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MODEL TESTS OF WAVE RUN-UP FOR HURRICANE PROTECTION PROJECT

by

Rudolph P. Savage
Research Division, Beach Erosion Board

Studies of protection for coastal areas against erosion damages and lowland flooding associated with the extreme high tide levels and wave action accompanying hurricanes are currently being made in several of the coastal divisions of the Corps of Engineers. A feasible method for providing this type of protection is the restoration or creation of an adequate dune barrier and protective beach by placement of suitable sand fill. In connection with a current investigation of this problem for an area which is exposed to the Atlantic Ocean it was necessary to evaluate the wave run-up to be expected for certain beach and dune conditions. In order to evaluate the wave run-up, an undistorted 1:20 scale fixed-bed model of a representative section of the proposed fill and nearshore topography of the area was built in a wave tank at the Beach Erosion Board and the wave run-up was measured with tide and wave conditions believed to be representative of storm and/or hurricane design conditions for the area.

Since these tests give information on one of the problems associated with hydraulic fill design, the data from the tests are being presented for possible use in other problems of this type. It is realized that the conditions encountered in other problems may differ from those given here, but it is believed that the data may have application in some cases.

The set-up for the model tests, shown in Figure 1, was installed in a wave tank 96 feet long, 2 feet deep, and 1½ feet wide. The average profile for the offshore section (see Figure 1) was obtained from a hydrographic survey made in the study area in 1955 and 1956, and was derived by using the intersection of mean sea level with the beach profile as a reference point in both the horizontal and vertical directions and then averaging selected measurements for all profiles except those in the immediate vicinity of an inlet. Assuming that within a short time after completion of the fill a similar profile would be formed (although located some distance seaward of the original profile), the beach and nearshore topography was constructed in the wave tank to a configuration as near that of the average profile as practicable using straight-line shapes. A dune slope of 1:10 approximates the present average value for existing dunes. The offshore bar in the tank model was composed of fine sand covered with a ½-inch layer of smooth concrete. The 1:20 beach foreshore slope, 1:10 and 1 on 5 dune slopes, and the berm of the beach were constructed of 3/4-inch plywood.

Preliminary to the actual testing, wave generator settings were determined which would give the desired wave heights seaward of the

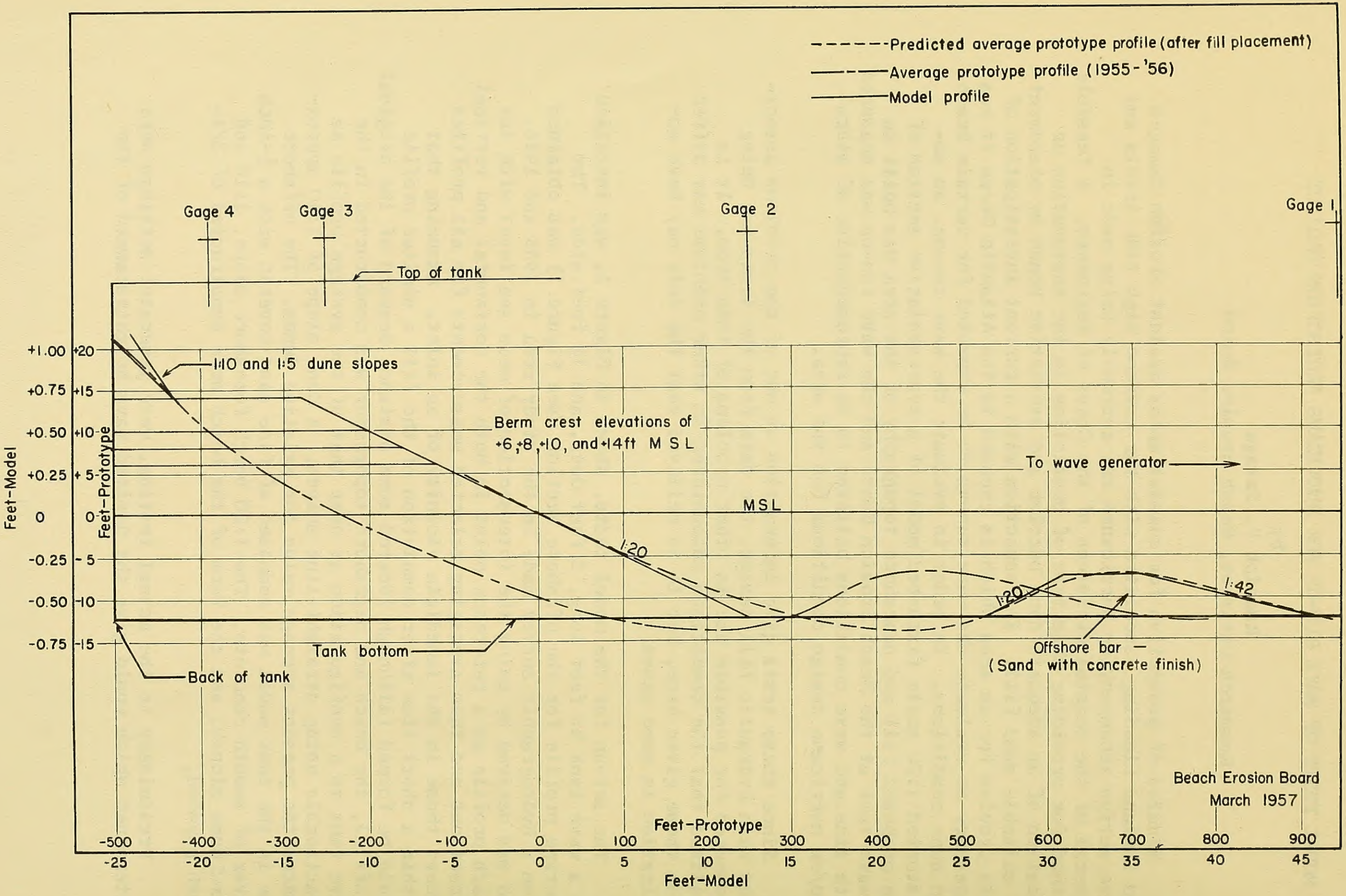

FIGURE I. MODEL SET-UP FOR TESTS OF WAVE RUN-UP

bar for all water depths and wave periods. Wave heights were measured during the tests at the positions indicated in Figure 1.

Prototype water elevations of +9 feet and +6 feet MSL were tested with prototype wave periods of 8 and 12 seconds and wave heights ranging from 6 to 12 feet, prototype. All of these conditions were tested with berm crest elevations of +6, +8 and +10 feet MSL, berm widths of 50 feet and 150 feet, and dune slopes of 1:10 and 1:5. These test conditions and results are given in Tables 1 through 6. Later computations indicated that it was possible to have a surge peak of +14 feet MSL in the area, and so further tests were made using this water level with berm elevations of +10.0 and +14.0 feet MSL, a dune slope of 1:5, and slightly different wave heights. The latter test conditions and results are shown in Tables 7 and 8.

The average wave run-up (determined by averaging the run-up for 9 to 15 waves disregarding the first 2 or 3 waves) is given in Tables 1 to 7. These values were obtained by comparing the run-up for two runs and; if agreement was good, no further runs were made. If agreement was not good, further runs were made to give a more representative average value.

In most of the tests the average run-up for the two runs agreed very well, differences being on the order of 0.0 to 0.2 foot (prototype). For these runs, the highest average run-up observed is given in Tables 1 to 7 if the difference was 0.1 foot, and the average value of the two runs is given if the difference was 0.2 foot. If the difference between the average run-up for the first two runs differed by more than 0.2 foot, the most consistent value of run-up is given. (Only 2 runs differed by 0.4 foot and 4 runs by 0.3 foot.) Wave heights were the same for all sets of runs.

For the data shown in Table 8, the run-up values measured were erratic. Since the reason for this erratic variation was unknown, it was believed that the average of the highest 1/3 of the run-up values would be more representative than the average run-up value. Consequently both the average run-up and the average of the highest 1/3 of the run-up values are given in Table 8.

Waves breaking on the 1:20 beach slope and moving across the beach berm caused a layer of water to stand on the berm. This stand or set-up is given as "water set-up at toe of dunes" (Tables 1 through 8). When this set-up is added to the "still water depth over the berm" (Tables 1 through 8), the "active water depth over the berm at toe of dunes" is obtained.

The footnote "Breaks over bar" in Tables 1 to 7 designates any energy dissipation by turbulence as the wave passed over the bar and does not necessarily indicate a plunging breaker. It may refer to a momentary "whitecap" or a "spilling" of the wave over a longer period of time.

Calculations based on the solitary wave theory and empirical curves for breaker depth versus deep water wave steepness[1] indicate that none of the waves used in the tests with a 14-foot berm elevation should have broken in the 21.5 feet of water over the bar. For example, the calculations give a breaking depth of approximately 16 feet for the wave of 12-foot height and 8-second period and a breaking depth of approximately 20 feet for the 12-foot high, 12-second wave. These test waves did break however, and the most probable explanation is that the theoretical and empirical curves apply to the breaker depth for waves travelling up a continuous slope and may fail when the slope abruptly levels. This explanation is supported by the observation that the test waves did not necessarily break on the seaward slope of the bar, but frequently spilled briefly over the level top of the bar, or in some cases, after passing the bar.

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1. Beach Erosion Board, "Shore Protection Planning and Design", Beach Erosion Board Technical Report No. 4, June 1954, pp. 48-50.

TABLE 1

DATA FROM WAVE RUN-UP MODEL STUDY

Berm elevation +6.0 ft. MSL (+7.5 ft. MLW) Berm width 150 ft.

Water depth over bar (Ft.)	S.W.L. (feet above MSL)	T (Sec.)	Wave Heights (Ft.)				Dune slope	Run-up (Ft. above SWL)	Run-up (Ft. above MSL)	Still water depth over berm (Ft.)	Water set-up at toe of dunes (Ft. above SWL)	Active water depth over berm at toe of dunes (Ft.)
			Seaward of bar	Shoreward of bar	50 ft. from crest of berm	150 ft. from crest of berm						
16.5	+9.0	12	12.0*	6.4	2.0	1.7	1/10	3.7	12.7	3.0	1.2	4.2
		12	8.0	7.4	2.0	1.9	1/10	4.0	13.0	3.0	1.4	4.4
		8	10.0*	6.0	1.7	1.3	1/10	3.0	12.0	3.0	1.2	4.2
		8	8.0	7.6	1.6	1.4	1/10	3.2	12.2	3.0	1.4	4.4
		12	12.0*	6.4	2.4	2.2	1/5	5.7	14.7	3.0	1.6	4.6
		12	8.0	7.4	2.4	1.8	1/5	5.9	14.9	3.0	1.2	4.2
		8	10.0*	6.0	2.0	1.8	1/5	4.5	13.5	3.0	1.2	4.2
		8	8.0	7.6	2.0	2.0	1/5	4.7	13.7	3.0	1.2	4.2
13.5	+6.0	12	11.8*	4.6	1.0	0.8	1/10	2.5	8.5	0	2.0	2.0
		12	10.0*	4.8	0.6	0.5	1/10	2.4	8.4	0	2.0	2.0
		12	8.0*	5.2	0.8	0.6	1/10	2.4	8.4	0	1.4	1.4
		12	6.3	6.2	1.0	0.8	1/10	3.0	9.0	0	2.0	2.0
		8	8.0*	5.0	0.4	0.4	1/10	1.6	7.6	0	1.0	1.0
		8	6.0	6.0	0.5	0.4	1/10	1.9	7.9	0	1.2	1.2
		12	11.8*	4.6	0.8	0.6	1/5	2.9	8.9	0	1.5	1.5
		12	10.0*	4.8	1.0	0.6	1/5	2.9	8.9	0	1.3	1.3
		12	8.0*	4.8	1.0	0.6	1/5	3.1	9.1	0	1.1	1.1
		12	6.3	6.2	1.2	0.6	1/5	3.6	9.6	0	1.7	1.7
		8	8.0*	5.4	0.4	0.4	1/5	2.0	8.0	0	1.1	1.1
		8	6.0	6.0	0.4	0.4	1/5	2.1	8.1	0	1.3	1.3

* Breaks over bar

TABLE 2
DATA FROM WAVE RUN-UP MODEL STUDY

Berm elevation +6.0 ft. MSL (+7.5 ft. MLW)

Berm width 50 ft.

Water depth over bar (Ft.)	S.W.L. (feet above MSL)	T (Sec.)	Wave heights (Ft.)				Dune slope	Run-up (Ft. above SWL	Run-up (Ft. above MSL)	Still water depth over berm (Ft.)	Water set-up at toe of dunes (Ft. above SWL)	Active water depth over berm at toe of dunes (Ft.)
			Seaward of bar	Shoreward of bar	50 ft. from crest of berm	150 ft. from crest of berm						
16.5	+9.0	12	12.0*	6.4	2.2		1/10	4.0	13.0	3.0	1.2	4.2
		12	8.0	7.4	2.2		1/10	4.2	13.2	3.0	1.3	4.3
		8	10.0*	6.0	1.4		1/10	2.7	11.7	3.0	1.0	4.0
		8	8.0	7.6	1.6		1/10	2.6	11.6	3.0	1.2	4.2
		12	12.0*	6.4	2.4		1/5	6.1	15.1	3.0	0.6	3.6
		12	8.0	7.4	2.4		1/5	6.3	15.3	3.0	0.6	3.6
		8	10.0*	6.0	1.4		1/5	5.2	14.2	3.0	1.2	4.2
		8	8.0	7.6	2.0		1/5	5.0	14.0	3.0	1.2	4.2
13.5	+6.0	12	11.8*	4.6	0.8		1/10	3.1	9.1	0	1.3	1.3
		12	10.0*	4.8	0.6		1/10	3.0	9.0	0	1.5	1.5
		12	8.0*	5.2	0.8		1/10	3.0	9.0	0	1.6	1.6
		12	6.3	6.2	0.8		1/10	3.2	9.2	0	1.5	1.5
		8	8.0*	5.0	0.6		1/10	2.1	8.1	0	1.1	1.1
		8	6.0	6.0	0.4		1/10	1.8	7.8	0	1.1	1.1
		12	11.8*	4.6	0.6		1/5	3.1	9.1	0	1.3	1.3
		12	10.0*	4.8	1.0		1/5	3.2	9.2	0	1.3	1.3
		12	8.0*	4.8	1.2		1/5	3.5	9.5	0	1.7	1.7
		12	6.3	6.2	1.0		1/5	3.6	9.6	0	1.3	1.3
		8	8.0*	5.4	0.4		1/5	2.3	8.3	0	1.1	1.1
		8	6.0	6.0	0.4		1/5	2.1	8.1	0	0.9	0.9

*Breaks over bar

TABLE 3
DATA FROM WAVE RUN-UP MODEL STUDY

Berm elevation +8.0 ft. MSL (+9.5 ft. MLW)

Berm width 150 ft.

Water depth over bar (Ft.)	S.W.L. (feet above MSL)	T (Sec)	Wave heights (Ft.)				Dune slope	Run-up (Ft. above SWL)	Run-up (Ft. above MSL)	Still water depth over berm (Ft.)	Water set-up at toe of dunes (Ft. above SWL)	Active water depth over berm at toe of dunes (Ft.)
			Seaward of bar	Shoreward of bar	50 ft. from crest of berm	150 ft. from crest of berm						
16.5	+9.0	12	12.0*	6.4	1.0	1.2	1/10	3.3	12.3	1.0	1.6	2.6
		12	8.0	7.4	1.4	1.2	1/10	3.6	12.6	1.0	1.8	2.8
		8	10.0*	6.0	0.8	0.6	1/10	2.7	11.7	1.0	1.2	2.2
		8	8.0	7.6	0.8	0.8	1/10	2.7	11.7	1.0	1.4	2.4
		12	12.0*	6.4	1.6	1.0	1/5	4.1	13.1	1.0	1.4	2.4
		12	8.0	7.4	1.8	1.0	1/5	4.2	13.2	1.0	1.6	2.6
		8	10.0*	6.0	1.0	0.6	1/5	3.3	12.3	1.0	1.2	2.2
		8	8.0	7.6	1.0	0.6	1/5	3.4	12.4	1.0	1.2	2.2
13.5	+6.0	12	11.8*	4.6			1/10	Run-up very small, Waves barely get over berm of beach		-2.0		
		12	10.0*	4.8			1/10			-2.0		
		12	8.0*	5.2			1/10			-2.0		
		12	6.3	6.2			1/10			-2.0		
		8	8.0*	5.0			1/10	No run-up		-2.0		
		8	6.0	6.0			1/10			-2.0		
		12	11.8*	4.6			1/5	Run-up very small. Waves barely get over berm of beach.		-2.0		
		12	10.0*	4.8			1/5			-2.0		
		12	8.0*	4.8			1/5			-2.0		
		12	6.3	6.2			1/5			-2.0		
		8	8.0*	5.4			1/5	No run-up		-2.0		
		8	6.0	6.0			1/5			-2.0		

*Breaks over bar

TABLE 4

DATA FROM WAVE RUN-UP MODEL STUDY

Berm elevation +8.0 ft. MSL (+9.5 ft. MLW) Berm width 50 ft.

Water depth over bar (Ft.)	S.W.L. (feet above MSL)	T (Sec.)	Wave heights (Ft.)				Dune slope	Run-up (ft. above SWL)	Run-up (ft. above MSL)	Still water depth over berm (Ft.)	Water set-up at toe of dunes (ft. above SWL)	Active water depth over berm at toe of dunes (Ft.)
			Seaward of bar	Shoreward of bar	50 ft. from crest of berm	150 ft. from crest of berm						
16.5	+9.0	12	12.0*	6.4	1.6		1/10	4.0	13.0	1.0	1.8	2.8
		12	8.0	7.4	1.8		1/10	4.0	13.0	1.0	1.6	2.6
		8	10.0*	6.0	1.0		1/10	2.7	11.7	1.0	1.4	2.4
		8	8.0	7.6	0.8		1/10	2.5	11.5	1.0	1.2	2.2
		12	12.0*	6.4	1.4		1/5	4.2	13.2	1.0	1.2	2.2
		12	8.0	7.4	1.6		1/5	4.4	13.4	1.0	1.4	2.4
		8	10.0*	6.0	1.0		1/5	3.6	12.6	1.0	1.4	2.4
		8	8.0	7.6	1.0		1/5	3.5	12.5	1.0	1.4	2.4
13.5	+6.0	12	11.8*	4.6	Wave heights very small		1/10	2.6	8.6	2.0	2.3	0.3
		12	10.0*	4.8			1/10	2.5	8.5	-2.0	2.3	0.3
		12	8.0*	5.2			1/10	2.6	8.6	-2.0	2.3	0.3
		12	6.3	6.2	0.4		1/10	2.9	8.9	-2.0	2.4	0.4
		8	8.0*	5.0	0		1/10	No run-up		-2.0	<2.0	0
		8	6.0	6.0	0		1/10			-2.0	<2.0	0
		12	11.8*	4.6	Wave heights very small		1/5	2.9	8.9	-2.0	2.4	0.4
		12	10.0&	4.8			1/5	2.7	8.7	-2.0	2.4	0.4
		12	8.0*	4.8			1/5	2.9	8.9	-2.0	2.4	0.4
		12	6.3	6.2			1/5	3.0	9.0	-2.0	2.4	0.4
		8	8.0*	5.4	0		1/5	No run-up		-2.0	<2.0	0
		8	6.0	6.0	0		1/5			-2.0	<2.0	0

*Breaks over bar

TABLE 5
DATA FROM WAVE RUN-UP MODEL STUDY

Berm elevation +10.0 ft. MSL (+11.5 ft. MLW)

Berm width 150 ft.

Water depth over bar (Ft.)$_M$	S.W.L. (feet above MSL)	T (Sec.)	Wave heights (Ft.)				Dune slope	Run-up (ft. above SWL)	Run-up (ft. above MSL)	Still water depth over berm (Ft.)	Water set-up at toe of dunes (ft. above SWL)	Active water depth over berm at toe of dunes (Ft.)
			Seaward of bar	Shoreward of bar	50 ft. from crest of berm	150 ft. from crest of berm						
16.5	+9.0	12	12.0*	6.4	0.6	0.4	1/10	2.7	11.7	-1.0	2.4	1.4
		12	8.0	7.4	0.6	0.4	1/10	2.8	11.8	-1.0	2.4	1.4
		8	10.0*	6.0	very small	0.4	1/10	2.4	11.4	-1.0	2.0	1.0
		8	8.0	7.6	very small	very small	1/10	2.1	11.1	-1.0	1.8	0.8
		12	12.0*	6.4	0.6	0.4	1/5	3.0	12.0	-1.0	2.2	1.2
		12	8.0	7.4	0.6	0.4	1/5	3.1	12.1	-1.0	2.2	1.2
		8	10.0*	6.0	very small	very small	1/5	2.4	11.4	-1.0	2.0	1.0
		8	8.0	7.6	very small	very small	1/5	2.2	11.2	-1.0	1.8	0.8
13.5	+6.0	12	11.8	4.6			1/10					
These conditions		12	10.0*	4.8			1/10					
not tested since		12	8.0*	5.2			1/10					
waves would not		12	6.3	6.2			1/10					
get over crest		8	8.0*	5.0			1/10					
of berm		8	6.0	6.0			1/10					
		12	11.8*	4.6			1/5					
		12	10.0*	4.8			1/5					
		12	8.0*	4.8			1/5					
		12	6.3	6.2			1/5					
		8	8.0*	5.4			1/5					
		8	6.0	6.0			1/5					

*Breaks over bar

TABLE 6
DATA FROM WAVE RUN-UP MODEL STUDY

Berm elevation +10.0 ft. MSL (+11.5 ft. MLW)

Berm width 50 ft.

Water depth over bar (Ft.)	S.W.L. (Feet above MSL)	T (Sec.)	Wave heights (Ft.)				Dune slope	Run-up (ft. above SWL)	Run-up (ft. above MSL)	Still water depth over berm (Ft.)	Water set-up at toe of dunes (ft. above SWL)	Active water depth over berm at toe of dunes (Ft.)
			Seaward of bar	Shoreward of bar	50 ft. from crest of berm	150 ft. from crest of berm						
16.5	+9.0	12	12.0*	6.4	0.6		1/10	3.2	12.2	-1.0	2.2	1.2
		12	8.0	7.4	0.6		1/10	3.2	12.2	-1.0	2.2	1.2
		8	10.0*	6.0	very small		1/10	2.5	11.5	-1.0	1.8	0.8
		8	8.0	7.6	very small		1/10	2.2	11.2	-1.0	1.8	0.8
		12	12.0*	6.4	0.4		1/5	3.0	12.0	-1.0	2.2	1.2
		12	8.0	7.4	0.6		1/5	3.3	12.3	-1.0	2.2	1.2
		8	10.0*	6.0	very small		1/5	2.4	11.4	-1.0	2.0	1.0
		8	8.0	7.6	very small		1/5	2.2	11.2	-1.0	1.8	0.8
13.5	+6.0	12	11.8*	4.6			1/10					
These conditions		12	10.0*	4.8			1/10					
not tested since		12	8.0	5.2			1/10					
waves would not		12	6.3	6.2			1/10					
get over crest		8	8.0*	5.0			1/10					
of beach berm.		8	6.0	6.0			1/10					
		12	11.8*	4.6			1/5					
		12	10.0*	4.8			1/5					
		12	8.0*	4.8			1/5					
		12	6.3	6.2			1/5					
		8	8.0*	5.4			1/5					
		8	6.0	6.0			1/5					

*Breaks over bar

TABLE 7
DATA FROM WAVE RUN-UP MODEL STUDY

Berm elevation +10 ft. MSL (+11.5 ft. MLW)

Dune slope 1/5

Water depth over bar (Ft.)	S.W.L. (Feet above MSL)	T (Sec.)	Wave heights (Ft.)				Run-up (ft. above SWL)	Run-up (ft. above MSL)	Still water depth over berm (Ft.)	Water set-up at toe of dunes (ft. above SWL)	Active water depth over berm at toe of dunes (Ft.)
			Seaward of bar	Shoreward of bar	50 ft. from crest of berm	150 ft. from crest of berm					
						50 ft. Berm Width					
21.5	+14.0	8	12.0*	9.4	2.8		6.3	20.3	4.0	1.6	5.6
		8	10.4	10.1	2.6		6.0	20.0	4.0	1.6	5.6
		8	8.0	7.7	2.6		5.9	19.9	4.0	1.2	5.2
		8	1.7	1.6	1.6		4.2	18.2	4.0	0.0	4.0
		12	14.0*	9.0	2.0		7.6	21.6	4.0	1.4	5.4
		12	12.0*	10.0	1.8		6.9	20.9	4.0	1.4	5.4
		12	11.2	10.0	1.8		7.0	21.0	4.0	1.4	5.4
		12	8.0	7.2	2.0		6.0	20.0	4.0	1.0	5.0
						150 ft. Berm Width					
21.5	+14.0	8	12.0*	9.4	2.6	2.4	6.7	20.7	4.0	1.4	5.4
		8	10.4	10.1	2.2	2.4	6.2	20.2	4.0	1.6	5.6
		12	14.0*	9.0	3.0	1.8	7.3	21.3	4.0	1.4	5.4
		12	11.2	11.2	2.8	1.8	7.3	21.3	4.0	1.6	5.6
		12	8.0	7.2	2.6	1.6	6.7	20.7	4.0	1.2	5.2

*Breaks over bar

TABLE 8
DATA FROM WAVE RUN-UP MODEL STUDY

Berm elevation +14.0 ft. MSL (+15.5 ft. MLW)

Dune slope 1/5

Water depth over bar (Ft.)	S.W.L. (Feet above MSL	T (Sec.)	Wave heights (Ft.) Seaward of bar	Wave heights (Ft.) Shoreward of bar	Wave heights (Ft.) At toe of dune	Run-up above SWL (Average) (Ft.)	Run-up above SWL (Average Highest 1/3 (Ft.)	Run-up above MSL (Average) (Ft.)	Run-up above MSL (Average Highest 1/3) (Ft.)	Still water depth over berm (Ft.)	Water set-up at toe of dunes (Ft. above SWL)	Active water depth over berm at toe of dunes(Ft.)
						50 Ft. Berm Width						
21.5	+14.0	8	12.0*	9.4	0.8	3.9	4.4	17.9	18.4	0	2.0	2.0
		8	10.4	10.1	1.0	3.6	4.1	17.6	18.1	0	1.6	1.6
		8	8.0	7.7	0.8	3.2	3.8	17.2	17.8	0	1.6	1.6
		12	14.0*	9.0	1.1	4.4	4.7	18.4	18.7	0	1.8	1.8
		12	12.0*	10.0	1.0	3.8	4.4	17.8	18.4	0	1.8	1.8
		12	11.2	10.0	1.0	3.6	4.3	17.6	18.3	0	1.8	1.8
		12	8.0	7.2	1.0	3.2	3.7	17.2	17.7	0	1.6	1.6
					No berm width - toe of dune at crest of beach berm							
										(at toe of dune)	(at toe of dune)	(at toe of dune)
21.5	+14.0	8	12.0*	9.4	1.4	3.9	4.4	17.9	18.4	0	2.0	2.0
		8	10.4	10.1	1.2	3.3	3.9	17.3	17.9	0	1.6	1.6
		8	8.0	7.7	1.2	3.3	3.7	17.3	17.7	0	1.6	1.6
		12	14.0*	9.0	4.2	4.9	5.3	18.9	19.3	0	2.4	2.4
		12	12.0*	10.0	3.6	4.1	4.4	18.1	18.4	0	2.0	2.0
		12	11.2	10.0	3.8	4.0	4.5	18.0	18.5	0	2.0	2.0
		12.	8.0	7.2	3.6	3.4	3.6	17.4	17.6	0	1.8	1.8

*Breaks over bar

WAVE REFRACTION PLOTTER

by
R. Q. Palmer, Chief, Civil Works Branch
Honolulu Area, San Francisco District
U. S. Army Corps of Engineers

The Honolulu Area of the San Francisco District, Corps of Engineers, recently prepared design analyses for several projects, each requiring a series of refraction diagrams. These diagrams were constructed in accordance with instructions contained in Technical Report No. 4 of the Beach Erosion Board entitled "Shore Protection Planning and Design.

The first series of refraction diagrams was constructed using a template,[1] illustrated on figure 17 of Technical Report No. 4. The template is a great improvement over previous devices used and it can be used with charts of any scale. Nevertheless, the work was time consuming and occasional errors were made due to slippage of the template or triangles used to orient the template. To overcome these problems, the template was adapted to a wave refraction plotter which could be attached to an ordinary drafting machine[2] (see Figure 1). Time trials by technicians trained on an equal period of time with both the template and the plotter showed that the production rate when using the plotter was more than twice that when using the template and fewer errors were made. Plotters can be fabricated for about $15 each in quantities of 2 or 3, but could be produced for about $10 each in amounts of 50 to 100.

Hydrographic work charts are prepared in the same manner as when using the template. The plotter is oriented for the selected deep water wave direction and the azimuth on the drafting machine is set at zero. The lower motion is then locked.

The upper motion of the drafting machine is freed and deflections are turned with the plotter. At all other times both motions are locked to fix the orientation of the plotter. The built-in pivot point is depressed with the palm of the hand when turning the deflection angles. A spring raises the pivot point when released. After each deflection angle is turned, the plotter is shifted slightly to bring the right angle-straightedge into position for extending the orthogonal. In the event that the orthogonal intersects the contours at angles less than 10 degrees, this fact is shown by angle markings on the plotter. Thus, guesswork or continual checking to determine when it is necessary to change either to or from the R/J method is eliminated. When a change to the R/J method is required, it is only necessary to read the azimuth on the drafting machine and by ready reference to the chart attached

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[1] After Arthur, Munk & Issacs, 1952

[2] When a drafting machine is used which has "stops" at 15-degree increments, the trigger on the "stop" should be depressed by placing a number of rubber bands around the control knob.

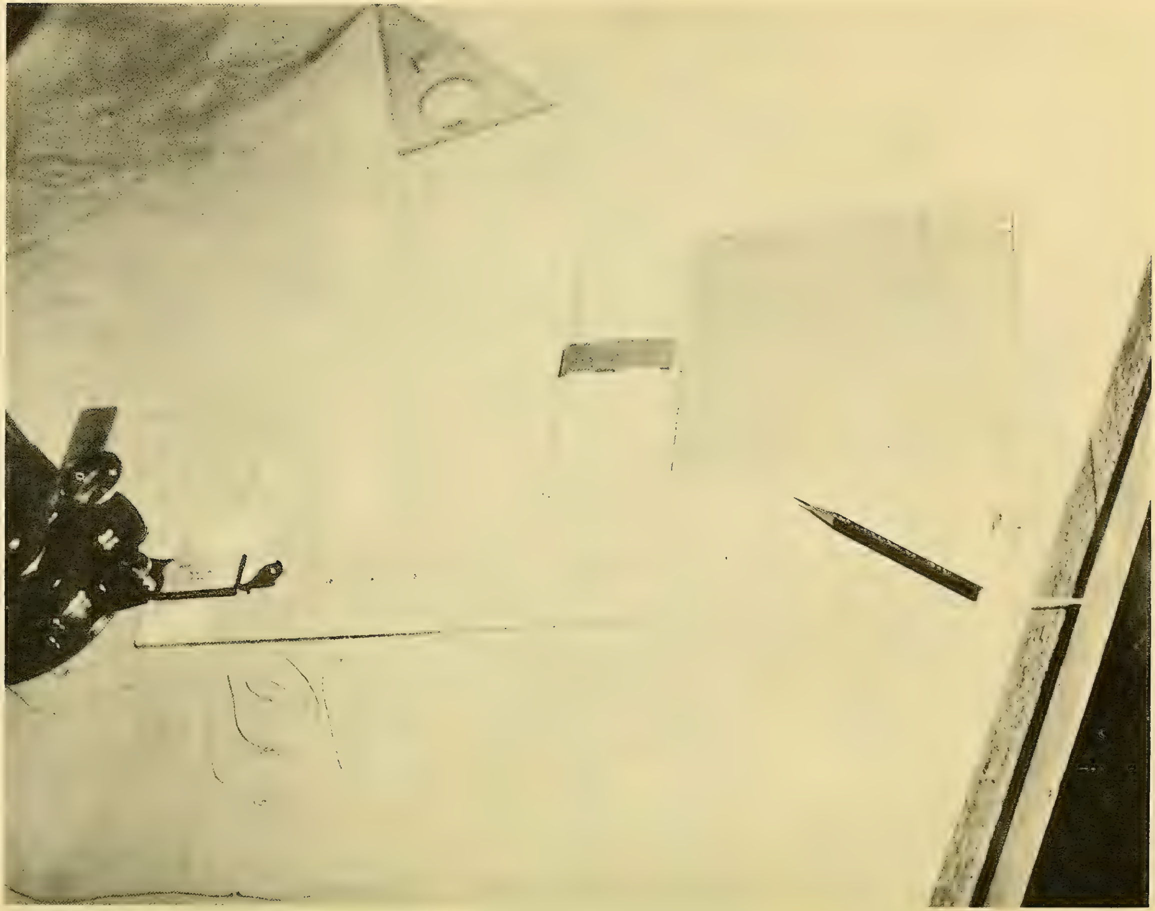

FIGURE I. WAVE REFRACTION PLOTTER ATTACHED TO DRAFTING MACHINE

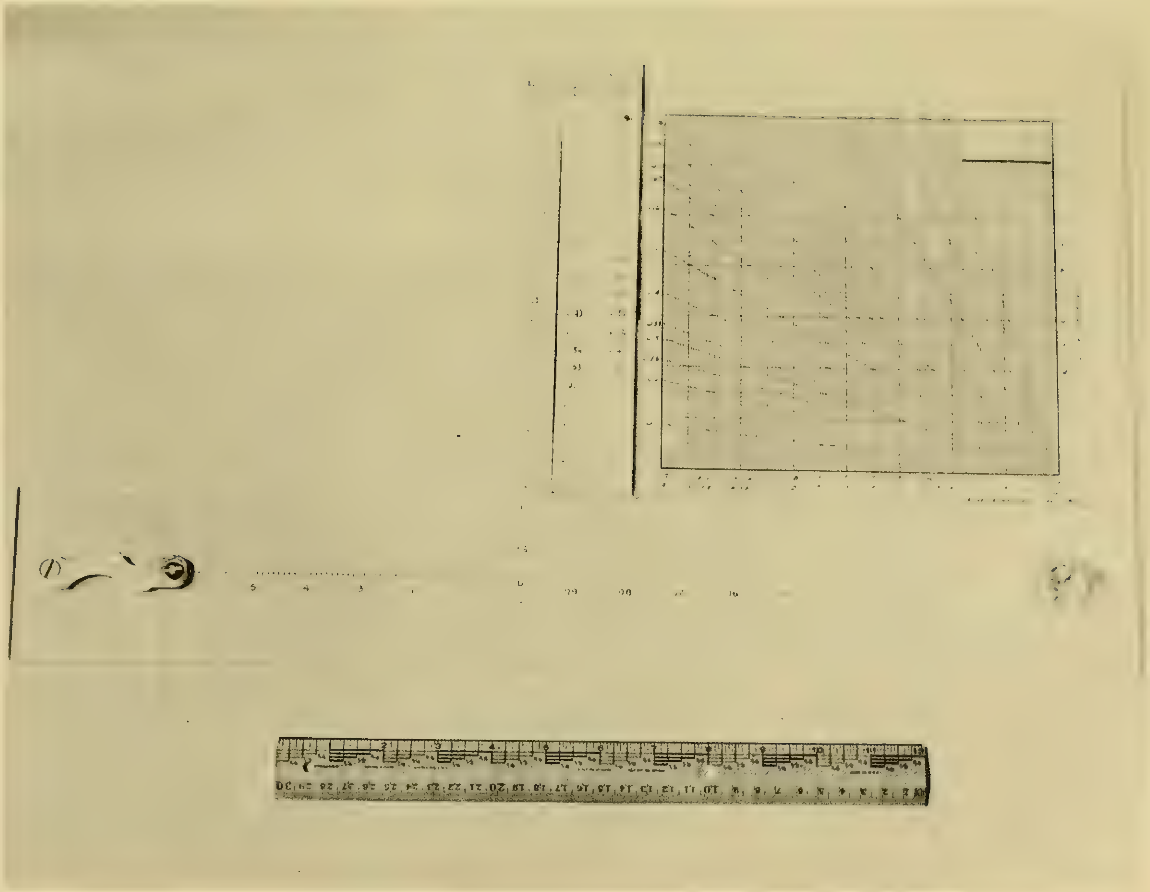

FIGURE 2. WAVE REFRACTION PLOTTER (CLOSE-UP)

to the plotter, the new azimuth is computed and set on the drafting machine to orient the plotter. Then the orthogonal is extended using the right angle-straightedge. Thus there is no loss of time in changing from one method to the other. In beginning the trace of each new orthogonal from deep water, the azimuth on the drafting machine is again set at zero degree; thus the plotter is properly oriented without other reference.

For convenience the proper table of values of C_1/C_2 may be attached to the plotter as shown on the photograph in Figure 2.

STATUS OF RESEARCH IN SHORE LINE PROTECTION

by
Joseph M. Caldwell
Chief, Research Division, Beach Erosion Board
Corps of Engineers, Department of the Army

This paper was prepared for and delivered at the August 1955 meeting of the American Society of Civil Engineers at Berkeley, California. Since this paper was not printed in the Proceeding of the American Society of Civil Engineers nor included as one of their separates, but does rather completely sum up the overall picture regarding the status of investigation and knowledge in the field of coastal engineering, it is reproduced here as information of probable interest to readers of the Bulletin.

Increasing use of the shores of the United States for recreation and seasonal housing developments has created greater public interest in the development of shore protection measures. Eroding shores have historically presented very interesting geological studies, but when the erosion exposes homes, roads, piers, parking lots, restaurants, hotels, railroads, canneries, beach clubs, and other miscellaneous constructions to undermining and wave damage, the study of shore line protection becomes of more practical and immediate concern.

The difficulty and expense of isolating and measuring the wave forces and shore characteristics which produce erosion militated against controlled research in this field until the economic pressure became great enough to clearly justify such action. Programmed research in this field began some fifteen years or so ago and has been accelerated since 1945 when Congress authorized the Beach Erosion Board of the Corps of Engineers to undertake research in this field. Prior to 1945, shore protection work in this country was done more or less on an empirical basis with relatively little funding to improve the status of knowledge.

The empirical approach by itself was not adequate due to the variations in the problem from place to place; it led not only to remarkable successes but also to disappointing failures. Present research is generally pointed toward establishing the fundamental concepts of shore processes on a quantitative basis and melding these findings with empirical knowledge in order to make shore protection a practical engineering science.

To describe the present status of research in shore line protection, the subject is broken down into three categories:

1. Wave action, i.e., the erosive force.
2. The reaction of the shore to the wave.
3. Improvement works to modify the reaction of the shore to the wave.

WAVE ACTION

Wave Generation. The generation, transmission, and decay of waves is a rather complex phenomenon. Classical wave theory (Airy, Gerstner) by itself does little to help the practicing engineer understand the wave action he encounters around the seacoast. Empirical formulae available before 1940 left much to be desired. Present practice finds two basic methods developed in this country for forecasting the generation and decay of waves. The first was developed during World War II to meet the need for accurate wave prediction in connection with amphibious landings; the method uses the concept of "significant waves" and is known as the Sverdrup-Munk method(1)* after its developers. The method was subsequently modified by Bretschneider(2).

More recently, a newer method of forecasting the generation and decay of wave trains has been developed which considers the entire component spectrum of the wave train in contrast to the Sverdrup-Munk definition of a single wave height and wave period as the significant wave. The more recent method was developed by Dr. Pierson and Dr. Neumann of the College of Engineering of New York University(3,3a).

Dr. Darbyshire of England has developed a method(3b) which also results in the prediction of a wave spectrum.

The Sverdrup-Munk method has seen widespread use over the past 10 years. The Pierson-Neumann method, being fairly recent and more complex in application, has not yet seen as extensive use. The Beach Erosion Board has had made a 3-year "hindcast" from synoptic weather charts using both methods(4,4a). A comparison of the results of these two hindcast studies shows a greater frequency of higher waves by the Sverdrup-Munk method; this difference may be attributed not only to differences in method and theory but also to the actual application of the methods (such as weather map analysis). Unfortunately there are not sufficient wave observations to check the relative accuracy of the two methods. Actually, both methods are completely dependent on accurate meteorological estimates of the velocity, extent, and duration of the winds which develop a given wave train. Further improvement in wave forecasting may accordingly have to await further developments in meteorology. Certainly, further improvements will have to await the accumulation of sufficient wave measurements to establish the accuracy of the prediction methods developed.

*Numbers in parentheses refer to list of references on pages 23 to 27.

Wave Refraction and Diffraction. Considerable work has been done on the theory of wave refraction and diffraction as a dominant factor in wave action in shoal waters(5,6,7). The basic concepts are simple; however, the application becomes very complex in an area of even moderately complex hydrography. Work is now under way at the University of California (Berkeley) under contract to the Beach Erosion Board to establish the degree of accuracy of present concepts for predicting wave refraction and diffraction over both simple and moderately complex hydrography. An estimation of the accuracy of existing methods and possibly simpler forms of their application may result from this study.

It is to be recognized that the above discussion on refraction applies to simple wave trains only; actually the complex wave trains of the ocean further complicate the application of refraction methods to actual problems. It is felt, however, that a quantitative understanding of the simpler aspects must precede the understanding of the more complex aspects.

Wave Dissipation. Waves may loose a considerable portion of their energy due to bottom friction and percolation as they move into the shallow water of the Continental Shelf. Theoretical studies of this have been made by Putnam and Johnson(8,9), laboratory studies have been made at the Beach Erosion Board(10), and field studies in the Gulf of Mexico by Texas A&M Research Foundation(11). Further progress on this phase of wave action will probably have to await a more comprehensive set of reliable field data showing the energy loss of wave trains travelling over the Continental Shelf.

Littoral Currents. Any obliquity of wave attack on the shore results in the generation of a littoral current along the shore face. For the most part these currents are more or less parallel to the shore but occasionally flow away from the shore as rip currents. Whatever their form, they have a considerable influence on shore processes. Due to the combination of complexities of wave action and irregularities of hydrography that are found in most areas, the attempts to correlate littoral currents with the concurrent wave action have been somewhat disappointing. However, simultaneous observations of waves and littoral currents are being accumulated in increasing numbers, and some satisfactory correlation of the two should be expected within the next 3 or 4 years. Theoretical formulae are available (Putnam, Munk and Traylor(12)) which develop the alongshore littoral current velocities to be expected from a simple, uniform wave train; however, the complexities of the natural ocean wave trains complicate the application of the method to natural conditions. Some work has been done on the gaining of a quantitative understanding of rip currents (Shepherd)(13), but much remains to be done.

SHORE PROCESSES

Sand Movement. The movement of beach sands by wave action is poorly understood at the present time. However a number of studies have recently been completed(14,15,16,17.18) or are now under way which should serve in the development of a quantitative understanding of this phenomenon. A laboratory study at Massachusetts Institute of Technology(19) ddals with the sorting action of waves on beach sands which results in the pulling of the fines into deep water; a laboratory study at the University of California(20) deals with the problem of fundamental mechanics of movement of sand particles by wave action. A field study at Scripps Institution of Oceanography deals with the seasonal changes in bottom depths from the shore line out to about the 100-foot depth contour. These studies are sponsored by the Beach Erosion Board of the Corps of Engineers and should lay the foundation of a quantitative understanding of sand movement by wave action.

Littoral Drift. The mass alongshore movement of sand by wave action is generally the key to shore erosion problems as well as many inlet navigation problems. The actual design of a shore protection plan frequently hinges on a proper estimate of the net rate and direction of littoral drift. At present this rate can usually be estimated accurately only where some structure, such as a breakwater or jetty, has created an impoundment area which enables a volumetric computation to be made. Some method of accurately estimating the rate of drift where no impoundments have taken place is needed. Attempts have been made along this line both in the laboratory(17,21) and in the field(21a)(21b) by working out a correlation of net wave energy upcoast or downcoast with the resulting rate of littoral drift; the results are promising but as yet too fragmentary to justify adoption as a basis of engineering design.

Effect of Natural Features. The effects of promontories and of inlets and submarine canyons in modifying the littoral drift and thereby the shore processes have only recently been subjected to intensive study. Studies at Scripps Institution of Oceanography(22) have led to a definition of the effect of submarine canyons on shore processes. Studies at the University of California (Berkeley)(23) have shown that rocky promontories extending out into as much as 30 feet of water are not barriers to the alongshore drift of beach sand. Small-scale studies at the Waterways Experiment Station are demonstrating the influence of littoral drift on tidal inlets and of tidal inlets on shore processes. Analytical studies of actual inlets and estuaries are also being made to develop a quantitative understanding of the part that inlets play in shore processes. These particular studies are sponsored by the Beach Erosion Board and other agencies of the Corps of Engineers.

Geologists are devoting an increasing amount of attention to the inshore area and their contributions are helping to clarify many aspects of shore behavior.(24,25)

From the broad viewpoint, however, the activity in the shore zone at a tidal inlet or the mouth of a large river presents a very complex-problem. Much work remains to be done toward gaining a better understanding of this feature of the shore.

IMPROVEMENT WORKS

Shore Structures. Shore structures are generally designed for one of two purposes: to protect the shore against erosion or to improve or shelter a navigation channel or harbor. Each design properly requires an answer to three basic questions:

a. What must be the dimensions of the structure to cause it to function as required?

b. What will be the forces imposed on the structure?

c. What materials can most reasonably meet the required strength and endurance characteristics?

The status of research for various types of shore structures relating to shore protection are given briefly in the following paragraphs.

Beach Groins. Research is actively under way at the Beach Erosion Board on the proper height, length, spacing and profile of groins. Little work is being done to measure the forces exerted on groins, required strength being estimated largely on the basis of experience. A report on the use of steel in shore structures including groins(26) was issued in 1952 and one dealing with the use of timber(26a) was issued in 1955. The Portland Cement Association has a booklet on concrete(27) and the Asphalt Institute may possibly issue one in the near future. Recent studies at the Waterways Experiment Station for the Bureau of Yards and Docks on the stability of rock mounds have added greatly to our knowledge of the stability of rock structures in wave action(28).

Seawalls. The influence of the shape of seawalls on wave run-up and quantity of overtopping water has been the subject of extensive tests for the Beach Erosion Board at the Waterways Experiment Station over the past three years. Vertical-face, stepped-face, curved-face, and sloping-face breakwaters have been tested over a wide range of wave conditions. Some of the results have been published(29) and a comprehensive report is expected to be published in a year or so by the Beach Erosion Board. Work still needs to be done on erosion at the toe of seawalls. At least two exploratory studies(30,31) have been made recently of the wave forces developed against seawalls; however much additional work is needed before a quantitative understanding is gained. Studies of the suitability of materials for use in seawalls are essentially the same as those mentioned for "beach groins" in the preceding paragraph.

Artificial Beaches. The establishment of beaches for recreational and protective purposes has come into widespread use in the last fifteen years. The protective effectiveness and stability of these beaches depends on the beach width, berm height, and grain size of the beach material. The degree to which a hurricane can cut back a beach and the effect of placing sand on the beach of a different size from that normally found thereon are samples of the questions that still need to be answered. Numerous studies(32,33) bearing on this problem of artificial beach design and construction have been made, but additional work must be done before the complete picture can be put together.

Breakwaters and Jetties. Though properly considered as navigation improvement structures, breakwaters and jetties sometimes have a significant effect on the adjacent shores. A better understanding of the short term as well as long term effects of these structures on the adjacent shore lines is needed as many of the most severe and costly erosion problems around out coasts have resulted from breakwater and jetty installations which interrupt the littoral drift of sand along the shore. A hydraulic model test of the broad aspects of this problem is planned as a continuation of generalized tests of tidal inlets now under way at the Waterways Experiment Station. The severe downdrift erosion sometimes resulting from the installation of these structures has led to the consideration of means of bypassing the impounded sand past the breakwaters or jetties in order to maintain the downdrift beaches; this is discussed in the next paragraph.

Sand Bypassing Methods. The obvious need at some jettied inlets and breakwaters to restore the normal littoral drift by bypassing the impounded sand has led recently to the consideration of methods of pumping the sand past the littoral obstruction. At least three bypassing projects are now in operation in the United States, one at Santa Barbara, California(34,35), one at South Lake Worth Inlet, Florida(21a), and one at Virginia Beach, Virginia. Three others are in a trial status or under design for Port Hueneme, California, Lake Worth Inlet, Florida, and Fire Island Inlet, New York. The design of these installations is somewhat problematical at present, the main question centering on what percentage of the net littoral drift can be reached by a fixed pumping plant, a trestle-mounted plant, or a conventional pipe-line dredge. It will probably be another four or five years before a definitive understanding of the best design to fit a given situation will be developed.

Key Problems. Two problems which, if solved, would make the design of shore protection works more reliable are:

1. Given the total net littoral drift, determine the quantity of sand being moved alongshore in various depth zones under selected wave conditions.

2. Determine how this distribution of alongshore drift with depth is modified by the installation of groins, jetties, breakwaters, and seawalls.

These two questions are inherent in all shore protection design problems and are particularly so in the design of fixed sand bypassing plants. An adequate quantitative method of solution to these two problems has yet to be devised, although the preceding portions of this paper show that individual facets of these two problems are being worked on and solved.

Recent Publications. In addition to the bibliographical references throughout this paper, there are six additional references of a rather comprehensive nature which, collectively, give a record of the present state of knowledge of shore processes and the design of shore protection works, together with a picture of the present research effort in this field. Five of those documents(36,37,38,39,40) are the published proceedings of the five coastal engineering conferences held under the auspices of the Council on Wave Research of the Engineering Foundation. Four of these conferences were held in the United States, one each on the shore of the Pacific (1950), the Gulf of Mexico (1951), the Atlantic (1952), and the Great Lakes (1953). The fifth conference was held in Grenoble, France in 1954. Collectively these five volumes contain 160 papers of interest to engineers and others interested in coastal engineering. The sixth document(41) is Technical Report No. 4 of the Beach Erosion Board, entitled "Shore Protection Planning and Design", published in 1954 and containing 242 pages plus six appendices. It is felt that the present paper would be incomplete if it failed to mention these six documents.

Summary. A review of the statements made in this paper and a study of the references will show that considerable progress has been made over the past 15 years in gaining a quantitative understanding of the fundamentals of shore protection processes. This has resulted from the combined efforts of the meteorologist, the oceanographer, the geologist, the applied mathematician, the research engineer, and the field engineer. Much work remains to be done; however, it is believed that another ten years will probably witness a firming up of our understanding and the placing of most shore protection design on a rational, quantitative basis.

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TESTS OF RIVER CREST STAGE GAGE UNDER WAVE ACTION

by
Thorndike Saville, Jr.
Assistant Chief
Research Division
Beach Erosion Board

ABSTRACT

Tests were made of a river crest stage gage to determine the effect of superficial wave action on the maximum stage recorded, with the idea that gages of this type might be adopted for measurements of maximum water elevations in estuaries and adjacent low-lying areas resulting from hurricane surges. Even slight wave action introduced considerable error in the maximum level recorded with the gage as originally planned, but it was found that the gage could be modified in such a way as to result in a maximum error of only a few tenths of a foot even with relatively high wave action.

The Beach Erosion Board was requested by the U. S. Weather Bureau to make some observations of the effect of wave action on the maximum water level recorded by a gage designed to provide a record of the peak stage reached in stream flow. If the response of the gage to such surface wave action was found to be slight, the Bureau recognized that the gage could possibly be used to record maximum stages reached by hurricane surges. The Hurricane Survey Coordinating Committee of the Corps of Engineers also felt that such a gage would be of considerable use in gathering hurricane and other storm surge data for design purposes, and was instrumental in having the tests continued with a modified gage. The gage initially supplied by the Weather Bureau was a U. S. Geological Survey river crest stage gage consisting of a 2-inch diameter plastic tube with a screw-on bottom cap having an effective bottom opening of 0.17 square inch, this opening being provided to allow an interchange of water between the outside and inside of the tube. The top of the tube was provided with a screw-on cap vented to permit escape of air. Inside the tube was a metal measuring stick which rested freely on the bottom cap, and extended nearly to the top cap. The inside of the tube had an effective area (pipe area minus rod area) of 3.07 square inches.

In field operation, the tubing would be of ordinary 2-inch metal pipe and would be mounted to a pier, piling, tree, building, or other suitable structure. Plastic tubing was used for the tests to permit easy visual observation of the effect of wave action. The crest stage in the field is recorded by means of a small quantity of ground burnt cork placed in the bottom of the tube.

As the water rises in the tube, the cork floats on the surface until the maximum stage is reached. As the water level starts to fall in the tube, a quantity of the burnt cork adheres to the measuring stick to leave a fairly permanent ring showing the maximum stage. The maximum stage may then be determined at any later date by removing the measuring rod and reading the level of the cork ring.

In the Board's tests the tube was mounted over a small wave flume, the bottom end of the tube extending 0.7 foot below the water surface; the water was 1.25 feet deep. Waves were then mechanically generated in the flume, and measurements of the amount of rise and fall of the water level within the measuring tube were made visually against a scale marked on the tube. Wave characteristics (height and period) were measured simultaneously using a standard two-probe resistance gage. A 1 on 6-slope, wave-absorbing, gravel beach was placed in the far end of the tank to prevent wave reflection. Wave periods tested in this tank ranged from 0.72 to 4.70 seconds, and wave heights from 0.03 to 0.70 foot. The measured values are shown in Figure 1 as a dimensionless plot of R/H vs H/T^2 where R is the water level rise inside the tube above the still water level, H is the measured wave height, and T is the wave period. The parameter H/T^2 is essentially the deep water wave steepness (H/L_o) being related by the equation $H/T^2 = (g/2\pi)(H/L_o)$; it is used rather than the actual steepness values because of its greater simplicity in computation. Actual measured values of water level rise ranged as high as 0.24 foot.

The plotted points show that the rise inside the tube varied from as little as one-tenth of the wave height for very steep short period waves to as much as 8 or 9-tenths of the height for lower steepness, longer period waves. The points were segregated as to wave period to see if this had an additional effect. As may be seen from Figure 1, actual period value appeared to have very little effect as long as it was above a value of slightly over 1 second; but for periods lower than this value an effect was observed.

To further test the possible effect of wave period, and to ascertain the validity of the model tested, certain tests were also made in the large prototype tank of the Board in conjunction with other tests being performed there. A 9-foot long tube was obtained and mounted near the wall of this tank, and measurements were obtained for essentially prototype waves of 5.6, 7.9 and 11.3-second periods ranging in height from 1 to 6 feet, in a water depth of 15 feet. Measurements were again obtained visually by reading against a scale mounted on the tubing. Measured rises inside the tube amounted to as much as 2.5 feet for a 6-foot wave. These prototype measurements are also shown in Figure 1. They may be seen to fall in with the much smaller scale data obtained in the small wave flume; they consequently show very little wave period effect to exist, other than that taken into account by variation in wave steepness.

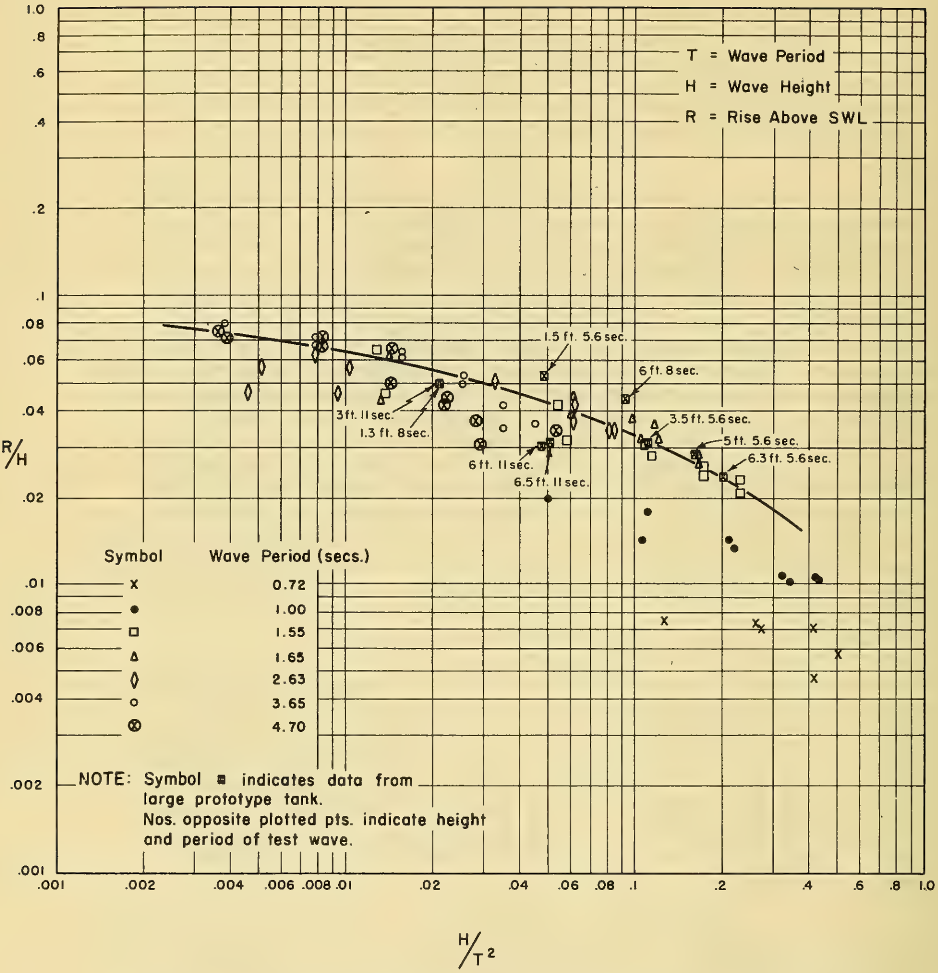


FIGURE I. WATER LEVEL RISE DUE TO WAVE ACTION FOR CREST STAGE GAGE WITH REGULAR OPENING

ERRATA SHEET

for

BULLETIN OF THE BEACH EROSION BOARD - VOL. 11, NO. 1, (JULY 1957)

Page 30 - Figure 1 of article titled "Tests of River Crest Stage Gage Under Wave Action" - correct ordinate scale for R/H values to range from 0.01 to 10.0 instead of 0.001 to 1.0.

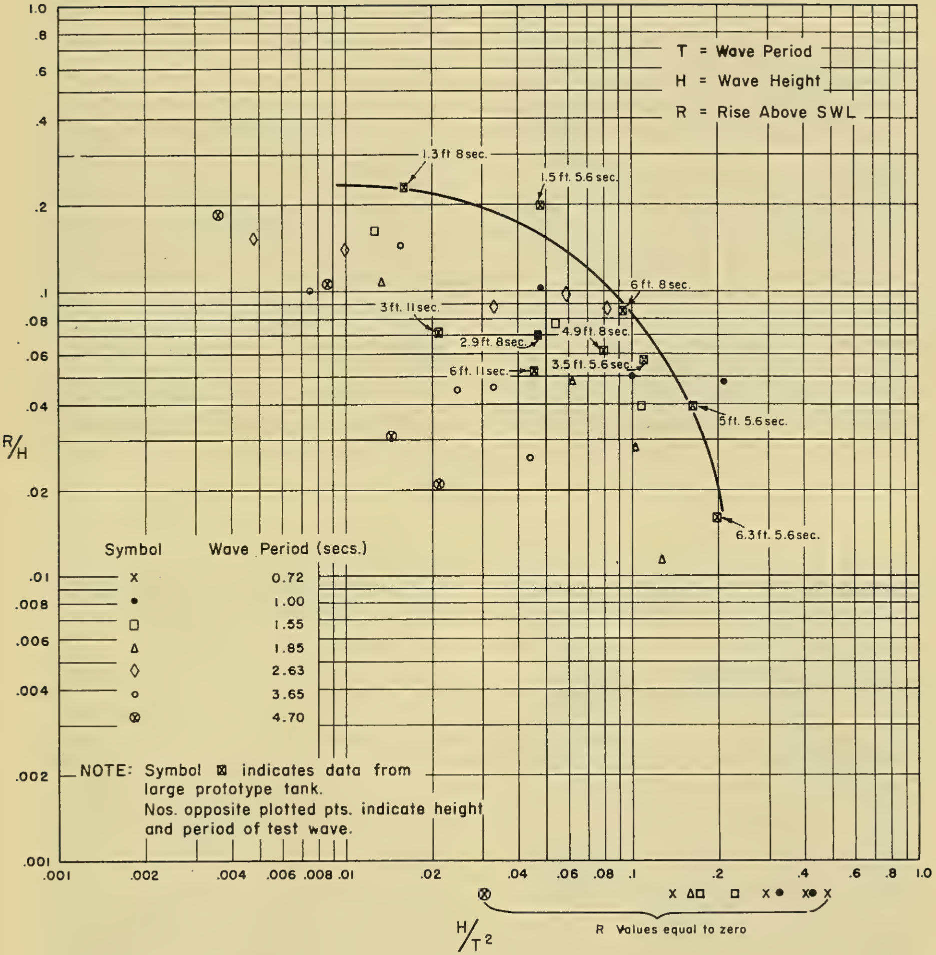
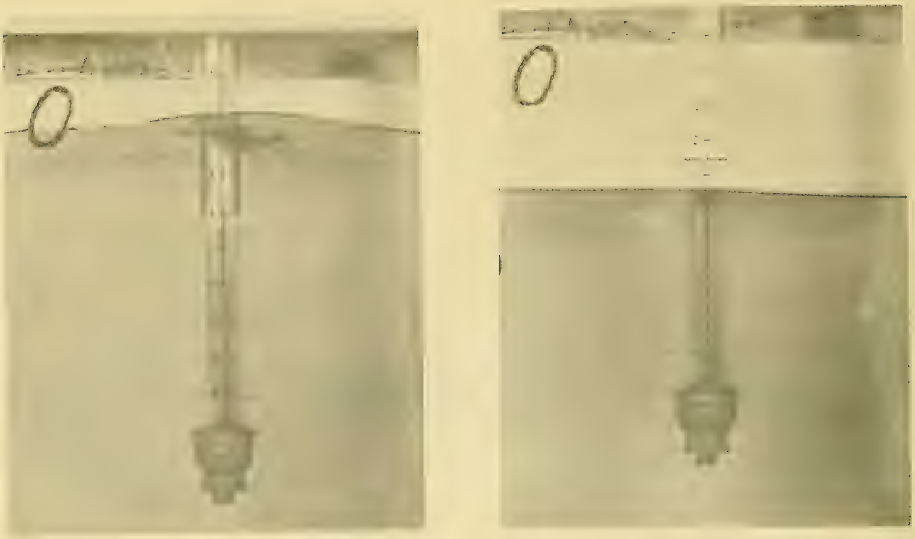
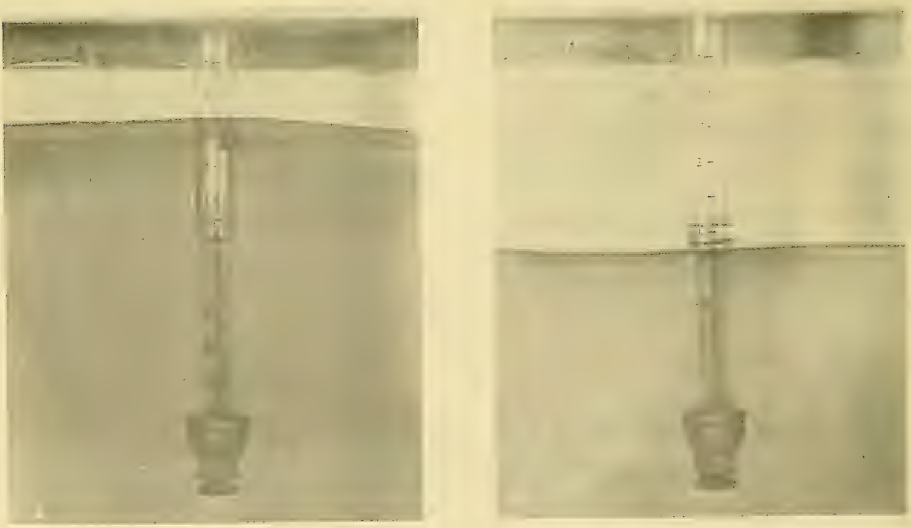


FIGURE 2. WATER LEVEL RISE DUE TO WAVE ACTION FOR CREST STAGE GAGE WITH REDUCED OPENING



a. Wave Crest Passing (Regular Opening)

b. Maximum Water Level Rise (Regular Opening)



c. Wave Crest Passing (Reduced Opening)

d. Maximum Water Level Rise (Reduced Opening)

FIGURE 3. PHOTOGRAPH OF CREST STAGE GAGE IN LABORATORY WAVE TANK

The magnitude of the response inside the tube to the wave fluctuations outside was deemed excessive, as leading to appreciable error in actual field measurements. Accordingly the gage was modified to reduce the ease of water interchange between the outside and inside of the tube by reducing the effective bottom opening to 0.040 square inch. The measurements were then repeated with this reduced section. Values obtained are shown in Figure 2. As may be readily seen, they were reduced considerably, the ride-up inside the gage never amounting to more than a quarter wave height, and most being less than a tenth. The highest value observed for tests in the small laboratory tank was 0.05 foot (for a 0.56-foot wave of 2.63-second period). For a number of test waves, particularly with the shorter periods, no measurable rise was recorded. In the prototype tank in only one case did the ride-up inside the gage exceed 0.3 foot (when it reached 0.5 foot for a 6-foot, 7.9-second wave) and most values were on the order of 0.2 foot.

The general degree of reduction in ride-up inside the tube with the reduced opening is also indicated in the photographs in Figure 3. Figures 3a and 3c show the gage as a wave crest passes by; note than while the original gage shows a water level rise of almost 0.1 foot (about one-third the wave height), the choked gage shows no rise. Figures 3b and 3d show the conditions at maximum water level rise inside the tube. For the original gage, this was 0.15 foot, as compared to only 0.02 foot in the choked gage. Note that, due to time lag in the response of the water level in the tube, the maximum rise always occurred considerably after the passage of the crest.

The lesser response to wave fluctuations of the water level inside the tube with the choked down opening was considered much more acceptable, and it is believed that most field installations by the Corps of Engineers since these tests have utilized gages with the lesser size opening. The effective area of 0.04 square inch approximates that of a 7/32-inch hole (0.038 square inch). Actually as many of the standard river crest gages supplied by the U. S. Geological Survey (through their Equipment Development Laboratory) come with a bottom-cap drilled with six 1/4-inch holes, a standard gage with five of these six holes closed up probably would give acceptable results for hurricane surge stage determination. Attention is called to the fact that a series of tests in Southern California ocean waters in the 1930's showed that if the bottom of the gage is continuously submerged, algae growth will close a 1/4-inch or smaller diameter hole. Holes of 1/2-inch in diameter were not closed by algae, however.

Although the tests were made to determine feasibility of using the gage to give a measure of the maximum hurricane high water stages, the results with the gage having the larger bottom opening would seem to indicate that the applicability of the general gage to measuring peak stream flood flows might also be reexamined to determine the possible effect of superficial wind waves and ship waves on these measurements.

PROGRESS REPORTS ON RESEARCH SPONSORED BY THE BEACH EROSION BOARD

Summaries of progress made during the past year on the several research contracts in force between universities or other institutions and the Beach Erosion Board, together with brief statements as to the status of some research projects being prosecuted in the laboratory of the Beach Erosion Board, are presented below. These summaries supplement and continue those contained in prior issues of the Bulletin.

I. University of California, Contract DA-49-055-eng-8. Sources of Beach Sand.

Seasonal sampling of Pt. Reyes beach and other beaches in the San Francisco area was continued. A report, "Changes in Configuration at Pt. Reyes Beach, California 1955-56" was published as Technical Memorandum No. 91. The report summarized the sand sample data at Pt. Reyes over a year's period, and presents one type of statistical analysis of the data. The report indicates that the beach is highly variable, with the sand ranging from relatively coarse in the fall to considerably less coarse in late winter. Also included in the report are data on the cut-and-fill of the beach area over a 2½-year period.

II. Massachusetts Institute of Technology, Contract DA-49-055-eng-16. Sorting of Beach Sand by Waves.

Additional data have been gathered on a flatter slope (1 on 25), particularly with regard to the incipient and equilibrium conditions and the mean net sediment velocity, primarily for waves in the 0.005 to 0.01 steepness range. A report on these tests is scheduled for submission shortly.

III. University of California, Contract DA-49-055-eng-17. Fundamental Mechanics of Sand Movement by Waves.

Additional data on the flow pattern near the bed under oscillating wave motion have been gathered by use of an oscillating bed. These data have been used in conjunction with a theoretical analysis to define coefficients for a description of bottom flow conditions for turbulent flow for the case of small amplitude waves and relatively deep water; a report dealing with this work was published as Technical Memorandum No. 97.

IV. University of California, Contract DA-49-055-eng-31. Wind Action Over Shallow Water.

Experimental work on the effect of bottom roughness in wind set-up and wave generation was performed using equally spaced strips of window screen serving to represent the roughness. Results of this study show that the roughness increased the set-up over smooth bottom conditions by as much as two times when the water depth was slightly above the roughness top, though decreasing it to a negligible amount when the water depth was about half way up on the roughness strips. Wave heights were not appreciably affected for large depths over the roughness tops, but were reduced for small depths; generally wave heights could be predicted adequately by using an "effective" depth equal to the depth over the roughness. A report was published as Technical Memorandum No. 95 of the Board. Funds for this study were provided by the District Engineer, Jacksonville, Florida.

V. University of California, Contract DA-49-055-eng-44. Laboratory Study of Wave Refraction.

Refracted wave characteristics were measured in the model tank for a series of wave, slope, and depth conditions. Measurements show that these conditions are being isolated. Data also shows that in general Snell's law applies (even for a vertical discontinuity in slope), although for certain conditions the waves considerably over-refract.

VI. Agricultural and Mechanical College of Texas, Contract DA-49-055-eng-45. Wave Statistics in the Gulf of Mexico.

Wave hindcast data were completed for five locations on the Gulf Coast for deep water and four shallow water depths (12,24,48, 96 feet) and were published as Technical Memoranda Nos. 85-89 of the Board. Also published (Technical Memorandum No. 84) was a description of the methods used in obtaining the data for the shallow water locations. A detailed analysis of a number of the more severe hurricanes in the Gulf was also prepared, and by using an energy coefficient, the relative severities of most of the hurricanes of this half century were obtained and compared (Technical Memorandum No. 98).

VII. University of Florida, Contract DA-49-055-eng-55. Analysis of Existing Data on Tidal Inlets.

Available data on certain tidal inlets in the United States and a few foreign localities have been reviewed. A detailed analysis of this data has been made for the inlets where sufficiently detailed data were found to exist. The purpose of the analysis is to derive, if possible, a mathematical expression interrelating the variables which govern the cross-sectional area and controlling depth of an inlet. An interim report has been submitted and a report is being prepared for publication. This study is administered by the Beach Erosion Board for the Corps of Engineers' Tidal Hydraulics Committee.

VIII. Agricultural and Mechanical College of Texas, Contract DA-49-055-Civ-eng-56-4. Estimation of Hurricane Surges.

A method was devised for taking into account the effect of bottom roughness and the resultant bottom shear stress on the flow of water into and through an estuary, and consequently the final water elevation around the shores of the estuary due to hurricane surge and for associated wind set-up (published in Technical Memorandum No. 93). Computations of surge distribution in Narragansett Bay have been made by applying this method using IBM computers, for several design hurricane conditions. These in turn are being utilized in conjunction with tests underway at the Waterways Experiment Station to obtain design criteria for the New England Division.

IX. Dr. W. C. Krumbein (Consultant). Study of Beach Sampling Methods.

A report was published (Technical Memorandum No. 90) comparing a number of (beach) sampling designs to indicate the factors involved in beach sampling for different purposes. In addition, data were obtained and analyzed on the behavior of artificially placed material in beach fill and nourishment projects to determine or verify design criteria for artificially placed beach material.

X. Waterways Experiment Station, Corps of Engineers.

a. Study of Interrelation of Tidal Inlets and Adjacent Beaches.

Testing was continued, expanding on studies made in previous years. A report was prepared (Technical Memorandum No. 94) describing the test results, primarily of two similar tests, one with a relatively deep lagoon and the other with a shallow lagoon. In both cases a down-beach inlet migration occurred, but where breakthroughs of the outer bar occurred several times with the deep lagoon, none occurred with the shallow lagoon until one was forced by superimposing a storm tide condition on the model. This study is supported jointly by the Beach Erosion Board and the Corps of Engineers Tidal Hydraulics Committee.

b. Study of Durability of Concrete.

A report summarizing the factors affecting concrete durability in Coastal Structures was published in Technical Memorandum No. 96. The publication also reports on the results of tests of specimens, and selected service records of specific prototype installations.

XI. Dr. John H. Davis - Contract for Study of Dune Stabilization.

A report on the study of shore protection by vegetation and planting on coastal dunes has been prepared and will be published as a Technical Memorandum. The report summarizes the mechanics of dune formation and the stabilization of these dunes by plants in the various coastal regions of the United States. It also presents data on the history of several continuing efforts to stabilize dunes by plants.

XII. Beach Erosion Board Staff.

(a) Wave Forces on Piles

Numerous measurements have been made of the wave forces on a vertical 12-inch pile with a 3-foot sensitive (instrumented) section. The pile is cantilevered from a frame at the top of the tank and may be raised or lowered to place the 3-foot sensitive section at various heights. Waves with periods ranging from 5 to 16 seconds, and heights ranging from 2 to 6 feet have been tested in water depths ranging from 7 to 15 feet. The forces on the 3-foot sensitive section ranged from 20 to 100 pounds for nonbreaking waves, and breaking waves produced forces of short duration as high as 1,000 pounds.

(b) Wave Run-up

Testing of eight smooth slopes ranging in steepness from 1 on 30 to a vertical wall has been completed and most of these slopes have been tested for the effect of roughness on wave run-up by roughening the slopes with five sand sizes ranging from 0.2 to 10 millimeters in median diameter. Also, most of the slopes have been tested for the effect of permeability on wave run-up for the same sands used in the roughness tests. The data show that slope roughness and permeability reduce wave run-up considerably below that observed for smooth slopes.

(c) Study of Sand Bypassing Operation at Port Hueneme

As a continuation of the observational program, a survey of the area was made in May 1957. The data taken before, during and after the Port Hueneme dredging to June 1955 have been analyzed and the report was published as Technical Memorandum No. 92. Computations to establish the volumetric changes between June 1955 and June 1956 have been made and according to these computations, over 2 million cubic yards of material was lost from downcoast beaches while the upcoast area gained in the order of 200,000 cubic yards of material. It is recognized that the major portion of the material from updrift is shunted into Hueneme Submarine Canyon by the west jetty and lost from the survey record.

(d) Laboratory Study of Effects of Groin Field on Littoral Drift Passing Field

Preliminary tests were made in the Shore Processes Test Basin to establish an equilibrium profile on which the groins will be placed. Installation of a new trapping and sand-moving system using eductors to transfer the trapped sand to the feeder beach area was completed. The new system has been used in several tests and works very well.

(e) Measurement of Suspended Material in Laboratory Wave Tanks

Sampling has been continued in the Shore Processes Test Basin (groin study) and the large outdoor wave tank. Analysis of the data is in progress and a draft of the report is being prepared.

(f) Wave Forecasting

Methods of forecasting for shallow water wave generation have been presented in Technical Memorandum No. 84 "Wave Forecasting Relationships for the Gulf of Mexico". Procedures include taking bottom friction into account. Formulas and techniques used are calibrated with respect to hurricane wind and wave data from Lake Okeechobee, Florida and some wave data for lighter winds of the Gulf of Mexico.

Analysis and comparison of the Bretschneider revised Sverdrup-Munk method with the spectrum method developed by Neumann and Pierson are being continued in an attempt to reconcile the two methods and establish more accurate values of the exponents used in the two methods.

The development of a joint distribution function for height and length, converted to a joint distribution function for height and period has been in progress. From this distribution function, a general form of the wave spectrum is being studied. A report on the above will be issued in the next year.

(g) Equilibrium Profile Tests

Equilibrium profiles were obtained in the prototype tank for the following wave conditions:

Wave Height	Wave Period
5.5 feet	11.33 seconds
5.3	5.60
5.0	3.75
2.0	16.0
5.3	16.0

Most of these wave conditions were also tested in the small wave tanks at 1:10 and 1:15 scales. The resulting data are to be used to establish profile scale relationships in movable bed beach models. A paper was prepared covering the results of the tests and is to be given at the meeting of the International Association of Hydraulic Research in Lisbon, Portugal.

(h) Hurricane Studies

The staff of the Board had been called upon to support the present hurricane study program of the Corps of Engineers. Considerable work has been done by the staff in developing and improving methods for estimating storm surge heights and wave heights under a variety of shoreline conditions. Wave forces and wave overtopping phenomena connected with seawall and dike design have also been studied.

(i) Recorded Wave Characteristics

Routine compilations were made of recorded wave characteristics at: Huntington Beach, California; Daytona Beach, Florida; Clearwater Beach, Florida; Palm Beach, Florida; Evanston, Illinois.

(j) Sand Movement Study at Moriches Inlet, Long Island, N. Y.

The report on sand movement at Moriches Inlet is being edited for publication.

(k) Reexamination of Beach Fill Projects

This includes a continuing program of reexamination of artificially nourished beaches to determine the effectiveness of the fill material within the beach zones, and to better establish the factors upon which to base the design characteristics of fill material. Reexamination report of a project at Virginia Beach, Virginia has been completed and is being edited for publication as a Technical Memorandum. Reexamination is now under way of projects at Plum Island, Massachusetts, West Haven, Connecticut and Presque Isle, Pennsylvania.

(l) Technical Report No. 4, "Shore Protection Planning and Design"

This involves a continuing study to improve and supplement present chapters of this publication. General corrections and addenda have been completed and will be disseminated when printed. Pertinent data from the Technical Memoranda on durability of concrete in coastal structures (TM 96) and on stabilization of dunes by vegetation will also be incorporated into Technical Report No. 4.

(m) Regional Studies

Data on the geomorphology and characteristics of littoral materials similar to that compiled for the Delaware Coast are being compiled for the South Shore of Long Island and being completed for the shore sector from Cape Henlopen, Delaware to Cape Charles, Virginia.

XIII. Publications

Technical Memoranda published by the Board to date are listed below. Those designated with an "x" are out-of-print, but others can be furnished on request to persons within the United States.

TM No.	Title and Date
x - 1	A Model Study of the Effect of Submerged Breakwaters on Wave Action, 1940
x - 2	Abrasion of Beach Sands, February 1942
x - 3	Shore Processes and Beach Characteristics, May 1944
x - 4	Surface Features of Coral Reefs, May 1944
x - 5	A Wave Method for Determining Depths Over Bottom Discontinuities, 1944
x - 6	An Ocean Wave Measuring Instrument, October 1948
x - 7	ShoreCurrents and Sand Movement on a Model Beach, September 1944
x - 8	Depths of Offshore Bars, July 1945
x - 9	Proof Test of Water Transparency Method of Depth Determination, July 1948
10	Experimental Steel Sheet Pile Groins, Palm Beach, Florida, 1948
x - 11	Reflection of Solitary Waves, November 1949
12	Durability of Steel Sheet Piling in Shore Structures, February 1952
13	Longshore Current Observations in Southern California, January 1950
14	Report on Beach Study in the Vicinity of Mugu Lagoon, California, March 1950
x - 15	Longshore Bars and Longshore Troughs, January 1950
x - 16	Accretion of Beach Sand Behind a Breakwater, May 1950
17	Test of Nourishment of the Shore by Offshore Deposition of Sand, May 1950
18	Rayleigh Disk as a Wave Direction Indicator, July 1950
x - 19	Submarine Topography and Sedimentation in the Vicinity of Mugu Submarine Canyon, California, July 1950
x - 20	Beach Cycles in Southern California, July 1950
21	The Interpretation of Crossed Orthogonals in Wave Refraction Phenomena, November 1950
x - 22	The Source, Transportation and Deposition of Beach Sediment in Southern California, March 1951
23	The Use and Accuracy of the Emery Settling Tube for Sand Analysis, May 1951
24	The Accuracy of Present Wave Forecasting Methods with Reference to Problems in Beach Erosion on the New Jersey and Long Island Coasts, April 1951
x - 25	The Slope of Lake Surfaces Under Variable Wind Stresses, November 1951
26	Sand Movement on the Shallow Inter-Canyon Shelf at La Jolla, California, November 1951
27	Wind Set-up and Waves in Shallow Water, June 1952
28	Sources of Beach Sand at Santa Barbara, California, as Indicated by Mineral Grain Studies, October 1952
29	Artificially Nourished and Constructed Beaches, December 1952
30	Annotated Bibliography on Tsunamis, February 1953
31	Laboratory Study of Wave Energy Losses by Bottom Friction and Percolation, February 1953

x out-of-print

32 Accuracy of Hydrographic Surveying in and Near the Surf Zone, March 1953
33 Laboratory Investigation of the Vertical Rise of Solitary Waves on Impermeable Slopes, March 1953
34 Development and Field Tests of a Sampler for Suspended Sediment in Wave Action, March 1953
35 Analysis of Moving Fetches for Wave Forecasting, March 1953
36 Wave and Lake Level Statistics for Lake Michigan, March 1953
37 Wave and Lake Level Statistics for Lake Erie, March 1953
38 Wave and Lake Level Statistics for Lake Ontario, March 1953
39 Areal and Seasonal Variations in Beach and Nearshore Sediments at La Jolla, California, March 1953.
40 The Mechanics of Deep Water, Shallow Water and Breaking Waves, March 1953
41 Laboratory Study of Equilibrium Profiles of Beaches, August 1954
42 A Study of Sand Movement at South Lake Worth Inlet, Florida, October 1953
x - 43 An Ocean Wave Spectra and a New Method of Forecasting Wind-Generated Sea, December 1953
x - 44 Coast Erosion and the Development of Beach Profiles, June 1954
x - 45 Modification of Wave Height Due to Bottom Friction, Percolation and Refraction, September 1954
46 Field Investigations of Wave Energy Loss in Shallow Water Ocean Waves, September 1954
47 Stability of Oscillatory Laminar Flow Along a Wall, August 1954
x - 48 Sand Movement by Waves, August 1954
49 Bore Hole Studies of the Naturally Impounded Fill at Santa Barbara, California, August 1954
x - 50 Statistical Significance of Beach Sampling Methods, August 1954
x - 51 Generation of Wind Waves over a Shallow Bottom, August 1954
52 Laboratory Study of Effect of Tidal Action on Wave-Formed Beach Profiles, December 1954
53 Laboratory Study of the Effect of Varying Wave Periods on Beach Profiles, September 1954
54 Laboratory and Field Tests of Sounding Leads, November 1954
55 North Atlantic Coast Wave Statistics Hindcast by Bretschneider Revised Sverdrup-Munk Method, December 1954
x - 56 An Electronic Wave Spectrum Analyzer and its use in Engineering Problems, October 1954
57 North Atlantic Coast Wave Statistics Hindcast by Wave Spectrum Method, February 1955
58 A Magnetic Tape Wave Recorder and Energy Spectrum Analyzer for the Analysis of Ocean Wave Records, July 1955
59 Laboratory Study of Shock Pressures of Breaking Waves, February 1955
60 Generalized Laboratory Study of Tsunami Run-up, January 1955
61 Laboratory Study of Wind Tides in Shallow Water, August 1955
62 Restudy of Test - Shore Nourishment by Offshore Deposition of Sand, Long Branch, New Jersey, November 1954.

63 A Study of Sediment Sorting by Waves Shoaling on a Plane Beach, September 1955
64 Laboratory Data on Wave Run-up and Overtopping on Shore Structures, October 1955
65 Sand Variation at Point Reyes Beach, California, October 1955
66 Factors Affecting the Economic Life of Timber in Coastal Structures, December 1955
67 A Model Study of the Run-up of Wind-Generated Waves on Levees with Slopes of 1:3 and 1:6, December 1955
68 Wave Action and Sand Movement Near Anaheim Bay, California February 1956
x - 69 Wave Forces on Piles: A Diffraction Theory, December 1954
x - 70 The Effect of Fetch Width on Wave Generation, December 1954
71 Re-Analysis of Existing Wave Force Data on Model Piles, April 1955
72 Laboratory Study of the Generation of Wind Waves in Shallow Water, March 1955.
73 Graphical Approach to the Forecasting of Waves in Moving Fetches, April 1955
74 Water Surface Roughness and Wind Shear Stress in a Laboratory Wind-Wave Channel, May 1955
75 Mechanics of Bottom Sediment Movement Due to Wave Action, June 1955
76 Movement of Sand Around Southern California Promontories, June 1955
77 Behavior of Beach Fill at Ocean City, New Jersey, March 1956
78 Hurricanes Affecting the Coast of Texas, Galveston to Rio Grande, March 1956
79 Orbital Velocity Associated with Wave Action Near the Breaker Zone, March 1956
80 Model Study of Overtopping of Wind-Generated Waves on Levees With Slopes of 1:3 and 1:6, April 1956
81 Laboratory Study of Short-Crested Wind Waves, June 1956
82 Changes in Sand Level on the Beach and Shelf at La Jolla, California, July 1956
83 Approximate Response of Water Level on a Sloping Shelf to a Wind Fetch Which Moves Directly Towards Shore, June 1956
84 Wave Forecasting Relationships for the Gulf of Mexico, December 1956.
85 Wave Statistics for the Gulf of Mexico off Brownsville, Texas September 1956
86 Wave Statistics for the Gulf of Mexico off Caplen, Texas, September 1956
87 Wave Statistics for the Gulf of Mexico off Burrwood, Louisiana, October 1956
88 Wave Statistics for the Gulf of Mexico off Apalachicola, Florida, October 1956
89 Wave Statistics for the Gulf of Mexico off Tampa, Florida, October 1956

90 Relative Efficiency of Beach Sampling Methods, September 1956
91 Changes in Configuration of Point Reyes Beach, California, November 1956
92 Sand Bypassing at Port Hueneme, California, March 1957
93 Modification of the Quadratic Bottom-Stress Law for Turbulent Channel Flow in the Presence of Surface Wind-Stress, February 1957
94 Preliminary Report: Laboratory Study of the Effect of an Uncontrolled Inlet on the Adjacent Beaches, May 1957
95 Effect of Bottom Roughness on Wind Tide in Shallow Water, May 1957
96 Factors Affecting the Durability of Concrete in Coastal Structures, June 1957

BEACH EROSION BOARD TECHNICAL REPORT NO. 4

"SHORE PROTECTION PLANNING AND DESIGN"

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Since publication of this report (first announced in the BULLETIN, Vol. 9, No. 2), 1670 copies of the 2,000 copies made available for sale have been sold.

The remaining 330 copies are available for purchase from the Superintendent of Documents, U. S. Government Printing Office, Washington 25, D. C., at $2.25 per copy.

BEACH EROSION STUDIES

Beach erosion control studies of specific localities are usually made by the Corps of Engineers in cooperation with appropriate agencies of the various States by authority of Section 2 of the River and Harbor Act approved 3 July 1930. By executive ruling the costs of these studies are divided equally between the United States and the cooperating agencies. Information concerning the initiation of a cooperative study may be obtained from any District or Division Engineer of the Corps of Engineers. After a report on a cooperative study has been transmitted to Congress, a summary thereof is included in the next issue of this Bulletin. Summaries of reports transmitted to Congress since the last issue of the Bulletin and lists of completed and authorized cooperative studies follow.

SUMMARIES OF REPORTS TRANSMITTED TO CONGRESS

SACO, MAINE

The city of Saco is located near the northeast corner of York County, about 16 miles southwest of Portland, along the southerly portion of Saco Bay. Saco Bay, located in the southwesterly portion of the Gulf of Maine, is about 7 miles in length and has a maximum width of 3 miles. It lies between protruding rocky headlands of Prouts Neck on the north and Fletcher Neck south of the jettied mouth of Saco River. The jetties at the mouth of Saco River are part of an inactive Federal navigation project. There are four islands and numerous rocky shoals in the near offshore area of Saco Bay.

The study area comprises the southerly 2.5 miles of the shore of Saco Bay, extending in a general southerly direction from Goosefare Creek to the jettied mouth of Saco River. The city of Biddeford occupies the adjacent area to the south and the towns of Old Orchard Beach and Scarboro that to the north. The city of Saco has a summer population of approximately 12,000 and about 10,000 permanent residents. A dune area begins about 2,000 feet north of Saco River and extends thence northward to Goosefare Creek. The crests of the dunes backing the beach are at elevations of 19 to 22 feet above mean low water. A storm scarp ranging from about 2 to 6 feet in height exists along the face of the dunes. In the southerly 2,000-foot section of the study area adjacent to the Saco River north jetty, city streets are located on the crest of the beach and are within range of wave uprush during storms. In the 700-foot reach of this section along Surf Street, the shore has receded to the revetted embankment, and the beach is used without restriction as a public beach. However, the only sections of shore under public control within the city of Saco are street ends comprising about 10% of the shore frontage, for which the city has easements.

The principal problem has been erosion of the shore throughout the study area. This erosion has resulted in loss of protecting dunes and steepening of the nearshore area. A stone revetment along Surf Street has furnished a degree of protection to that street, but has relatively high maintenance costs. Northward of the revetment area, the principal damage has been a loss of sand from the beach. This loss has reduced the width of beaches, which are necessary to protect property from wave damage. The tides at Saco are semi-diurnal. At Old Orchard Beach, adjacent to Saco, the mean range is 8.8 feet and the spring range is 10.1 feet. Waves affecting the Saco shore approach from between east-northeast and east-southeast. There is a small southward predominance in direction of littoral drift, as indicated by past accretion at the Saco River north jetty. Lack of accretion in recent years appears to have been the result of passage of material over or landward of the inner end of the jetty.

The division engineer concluded that the most practicable method for protecting the city of Saco shore against property damage and dune recession is a comprehensive sand fill plan. The plan included the construction of a sand beach with a berm 50 feet wide at elevation 15 feet above mean low water, by artificial placement of approximately 570,000 cubic yards of suitable sand for the initial fill. The division engineer also concluded that the most practicable method for protecting only the shore between the north jetty at Saco River and Sunrise Avenue, where the principal damage has occurred, was a partial sand fill plan. For a beach with 50-foot berm width at elevation 15 feet above mean low water, he estimated this plan would involve placement of about 140,000 cubic yards of suitable sand for the initial fill. Maintenance of the beach fills would be by periodic placement of additional sand. The estimated average annual requirements, as estimated by the division engineer, were respectively 81,000 and 11,000 cubic yards for the comprehensive and partial fill plans. In order to reduce the loss of beach material around the inshore end of the Saco River north jetty, the jetty would be extended landward under either of the sand fill plans. The division engineer further concluded that although the existing revetment along Surf Street between Lower Beach Road and Sunrise Avenue is in fair condition, a heavier and tighter structure with lower maintenance costs could be provided by chinking the present revetment with smaller stone and placing additional large cover stones. After making an economic analysis of the considered protective plans, the division engineer concluded that none of the plans were economically justified by evaluated benefits and recommended no Federal project be adopted at this time by the United States for the protection of the shore of Saco, Maine. He further recommended that protective measures which may be undertaken by local interests, based upon their determination of economic justification, be accomplished in accordance with methods proposed and projects considered in his report.

The Beach Erosion Board concurred generally in the views and recommendations of the division engineer. However, the Board separately estimated the annual nourishment required to maintain the comprehensive and partial fill plans at 34,000 and 5,600 cubic yards respectively, and considered a beach berm width of 25 feet to be adequate to prevent erosion of the dunes or damage to the revetment along Surf Street. The Board also stated its belief that closure of the gap at the shore end of the Saco River north jetty should be considered for construction regardless of any other measures which may be undertaken for protection of the shore, and recommended that the closure be accomplished as soon as practicable with Federal funds, when such funds become available for maintenance of navigation structures. It also recommended that, subject to their own determination of economic justification, local interests consider adoption of plans of protection developed by the division engineer, especially that for closure of the gap at the shore end of the jetty, if this is not done by the Federal government, and the partial beach fill plan with either a 50-foot or 25-foot berm width, and that protection which may be provided by local interests be accomplished generally as proposed by the division engineer.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

CHATHAM, MASSACHUSETTS

The town of Chatham is located in Barnstable County at the southeast corner of Cape Cod, about 75 miles southeast of Boston. The mainland portion of the town is located on relatively high glacial deposits. Morris Island, an undeveloped glacial deposit south of the mainland, was connected to the mainland by a barrier beach for many years prior to 1938. A breach through this beach, initiated during the 1938 hurricane, was enlarged by storms in 1940 and 1944. Nauset Beach is a sand spit extending southward about 10 miles from the east side of the Cape Cod mainland in front of Pleasant Bay and the Chatham mainland and presently ending about 1/2 mile east of Morris Island. Monomoy Island is another extensive sand spit extending about 8 miles southward from Morris Island and separating Chatham Roads, the northeast corner of Nantucket Sound, from the Atlantic Ocean. Stage Harbor lying between the Chatham mainland and Morris Island is used for fishing and recreational purposes. Its entrance from Chatham Roads has been dredged under a Federal navigation project. Chatham is a popular summer resort area. The permanent population of the town of about 2,500 is greatly increased by summer residents and visitors. The shores of the town affected by the considered plans are privately owned.

The tides in the study area are semi-diurnal. The mean ranges in the ocean and Chatham Roads are respectively 6.5 and 4.3 feet. The maximum 10-year high waters at these locations are estimated to be respectively 10.2 and 5.9 feet. Under conditions at the time the study was made, due to the shoal entrance between Nauset Beach and Morris Island, the mean-tidal range immediately east of the breach was about 4.7 feet. The maximum 10-year high water was estimated to be 6.5 feet. The east shores of Cape Cod, Nauset Beach and Monomoy Island are exposed to waves generated in the unlimited fetch of the North Atlantic. Under conditions existing at the time of study the breach area is protected from heavy wave action by Nauset Beach. Beach material has been supplied to the shores of the study area by littoral transport along Nauset Beach from erosion of bluffs to the north. The heavy drift has formed the extensive sand spits of Nauset Beach and Monomoy Island. With Nauset Beach presently extending southward east of Morris Island, little or no sand was being supplied to the shore in the vicinity of the breach through the beach between the Chatham mainland and Morris Island. Stage Harbor has been subjected to shoaling by material eroded from the banks and bottom of the breach and oonnecting channel between the breach and the harbor.

The division engineer developed plans for preventing shoaling of Stage Harbor caused by the breach north of Morris Island and damage to shore property in the vicinity of the breach. He concluded that the most suitable plan for the protection of Stage Harbor from shoaling due to these causes is a sand dike closure. He further concluded that the harbor shoaling has not encroached upon the Federal navigation project and that the shore areas are privately owned, and therefore a Federal project for controlling or closing the breachway is not warranted. He recommended that local interests consider construction of the sand dike closure on a westerly alignment and that no project for this purpose be adopted by the United States at this time.

The Beach Erosion Board recommended that local interests consider the relative advantages and disadvantages of closure of the breach between the Chatham mainland and Morris Island by a sand dike, of a bulkhead navigation plan, and of dredging to relieve shoaling of Stage Harbor. In the event they should consider complete closure sufficiently advantageous, the Board recommended closure by a sand dike on the westerly alignment, generally in accordance with the plan developed by the division engineer. The Board also recommended that no project be adopted by the United States at this time for construction of works to close or improve the breach between the Chatham mainland and Morris Island.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

STATE OF CONNECTICUT - SAUGATUCK RIVER TO BYRAM RIVER - (AREAS 8 & 11)

Areas 8 and 11 of the State of Connecticut study comprise the shores of Long Island Sound between the mouth of Saugatuck River in Westport and the New York State line at Byram River. It includes the shores of the cities of Norwalk and Stamford and of the towns of Darien and Greenwich and a portion of the shore of the town of Westport, a total shore frontage of about 35 miles. Stamford near the middle of this shore area, is about 32 miles east of New York City. The shore area is extensively developed for year-round residential use. The permanent population of Westport, Norwalk, Darien, Stamford and Greenwich is about 215,000. There is little seasonal change in population. The principal publicly owned sections of shore in the study area are Calf Pasture Beach Park and Bailey Beach in Norwalk, Peartree Point and Weed Beach in Darien, Cove Island, Cummings Park, West Beach, Dyke Park and Southfield Park in Stamford, and Greenwich Point, Byram Park and Little Captain Island in Greenwich.

Long Island Sound is a tidal arm of the Atlantic Ocean. Tides are semi-diurnal, the mean range increasing from 7.0 feet at Saugatuck River to 7.4 feet at Greenwich. The spring ranges are respectively 8.3 and 8.7 feet. The maximum tide of record at Stamford was 13.8 feet above mean low water or 6.6 feet above mean high water. Tides 3 feet or more above mean high water occur about once a year. With a tidal stage of 3 feet above mean high water, the maximum height of breakers landward of the low water line is about 8 feet. Larger waves can reach the shore only during infrequent higher tides. Ocean swells entering Long Island Sound between Race Point and Little Gull Island probably have negligible effect on the shores of the study area near the west end of the sound. The waves of primary importance are those generated in the sound. Ordinary short storm waves cause littoral movement and offshore loss of beach material. The influence of swells is probably insufficient to cause appreciable return of material from offshore by wave action. Waves which cause the greatest movement of beach material are those from the east. The predominant direction of littoral drift is northward along shores aligned generally north and south, and westward along shores aligned generally east and west.

The study area is characterized by rocky headlands and headlands of unconsolidated glacial material. In a few localities wave-built bars or spits have been formed. The headlands formerly supplied material to the intervening beaches, but are now generally eroded to bed rock or otherwise protected. The supply of material has thus been reduced or eliminated, and consequently the beaches have slowly deteriorated. Groins have been found to be capable of causing minor accretion areas and stabilizing a narrow band along the upper portion of the beach in some sections, but the natural supply of material is insufficient for the formation of adequate protective beaches by groins alone. The building and maintenance of adequate beaches can be accomplished by artificial placement of sand. The rate of loss of fill can be reduced by groins.

The division engineer and Beach Erosion Board concluded that practicable plans which merit consideration for the protection and improvement of beaches within the study area are as follows:

(a) Calf Pasture Beach Park - East Shore, Norwalk. Widening approximately 2,200 feet of beach to a 125-foot width by direct placement of sand fill and lengthening two existing riprap groins to a 400-foot length;

(b) Calf Pasture Beach Park - South Shore, Norwalk. Construction of two impermeable groins to a 350-foot length;

(c) Cove Island, Stamford. Widening to a 125-foot width by direct placement of sand fill, 1,200 feet of the east shore of Cove Island and construction of an impermeable jetty 400 feet long at the east limit of the fill, (the fill material may be obtained by excavation of a wider and straighter entrance channel to Holly Pond in a manner that will reduce the swift currents close to Weed Beach);

(d) Cummings Park, Stamford. Widening to a 125-foot width in front of the existing bathhouse by direct placement of sand fill approximately 1,000 feet of the public bathing beach, lengthening of the existing groin to a 400-foot length, and raising the inshore end of the existing jetty;

(e) Greenwich Point, Greenwich. Widening, generally to a 125-foot width by direct placement of sand fill, approximately 2,800 feet of the public bathing beach;

(f) Byram Point, Greenwich. Landward extension and repairs to the existing jetty to make it impermeable to the passage of sand.

The division engineer and Beach Erosion Board also found that protection and improvement of the east shore at Calf Pasture Beach Park and shores at Cove Island, Cummings Park and Greenwich Point are justified by evaluated benefits. They further found that the nature and amount of benefits warrant Federal participation, and recommended subject to certain conditions, adoption of projects by the United States authorizing Federal participation by the contribution of Federal funds in an amount equal to one-third of the first costs of construction of the plans for those four publicly owned shores. They also recommended that protective measures which may be undertaken by local interests, based on their determination of economic justification, be accomplished in accordance with the methods proposed and projects considered in his report.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

SANTA CRUZ COUNTY, CALIFORNIA

Santa Cruz County is in the central portion of the coast of California, about 65 miles south of San Francisco. Its Pacific Ocean shore line, extending in a general northwest-southeast direction, is about 40 miles long. The coastal area is divided into three principal natural segments. From the northern county line to the west city line of Santa Cruz, the shore cliffs range from 30 to 250 feet in height. This segment has a northwest-southeast orientation. Shallow indentations and coves contain sandy beaches and dunes at the mouths of the short steep intermittent streams draining the mountain slopes. The second segment, comprising about 8 miles of shore to New Brighton Beach has a general east-west orientation and forms the northern boundary of Monterey Bay. Cliffs range from 30 to 75 feet in height. This segment comprises the principal problem area and is divided into five sections as follows:

a. West Cliff section extends from the west city limit of Santa Cruz to Cowells Beach. It consists of irregular eroding cliffs of soft shale without protective beaches.

b. Santa Cruz Harbor section comprises Cowells and Santa Cruz Beaches. These beaches have a general east-west orientation and are wide and stable. No erosion problem exists in this section.

c. Twin Lakes section includes two cliff portions separated by a barrier beach in front of Woods and Schwan Lagoons. The cliffs are subject to erosion by winter wave action, as the winter beaches are too narrow to afford adequate protection.

d. Del Mar section is similar to the Twin Lakes section. It has three cliff sections and two lagoons. The cliffs are eroded in the winter when beaches are inadequate to dissipate wave energy.

e. Opal Cliffs - Capitola section, extending from Soquel Point to New Brighton Beach, consists of two cliff sections separated by Capitola Beach at the mouth of Soquel Creek. No beaches exist in front of the cliffs, which are consequently subject to erosion.

The third segment of the study area lies on the east shore of Monterey Bay, extending in a generally southeastward direction to the mouth of Pajaro River. Sandy beaches, generally 50 to 200 feet wide at high water, are backed by bluffs or sand dunes ranging from 90 to 200 feet in height. The shore line in this segment is relatively stable. The cities of Santa Cruz and Capitola with a combined population of about 24,000 are in the central part of the county shore on the north shore of Monterey Bay. The populations of these communities increase about threefold in summer. Their ocean frontages, combined with the intervening county frontage, aggregate about 8 miles and comprise the principal problem area. Of this frontage about 42 percent is publicly owned or in process of acquisition by the State.

The tides on the ocean shore of the study area have a diurnal inequality, the mean diurnal range being 5.3 feet. The maximum tide is estimated to be about 8 feet above mean lower low water. Characteristic deep-water waves have significant heights ranging from 2 to more than 20 feet and periods ranging from 4 to 20 seconds. The mean height and period are 4 feet and 13 seconds. The prevailing direction of wave approach is from the northwest. Winter storm waves with heights of 10 feet or more usually approach from the southwest quadrant. Beach material is supplied to the shores of the problem area by littoral transport from shores to the north, by erosion of the bluffs and by stream deposits within the area. Wave forces having a predominant downcoast component are adequate for a substantial rate of littoral transport, so that available material is moved rapidly downcoast through the problem area along the north shore of Monterey Bay to the east shore of the bay. Reversals in transport direction occur along the northern shore of Monterey Bay, except in those sections having north-south or northeast-southwest orientations such as just north of Point Santa Cruz and the Opal Cliffs - Capitola section. In general, those sections have no beaches, but other sections having a northwest-southeast orientation, such as Santa Cruz, Twin Lakes and Del Mar Beaches, have adequate beaches in summer. Winter profile adjustments reduce the width of the latter two beaches, so that inadequate protective beaches remain in front of the bluffs. The estimated average rate of recession of the unprotected soft rock cliffs has been 1 foot annually.

The district and division engineers and the Beach Erosion Board concluded that the most suitable and economical plans of shore protection for the several sections of the problem area are as follows:

a. For the West Cliff, Del Mar and Opal Cliffs-Capitola sections, riprap seawalls;

b. For the Twin Lakes section, a beach fill with a minimum width of 120 feet along both the easterly and westerly portions, and two rubble-mound groins.

The district and division engineers and the Beach Erosion Board concluded that the plans for the West Cliff and Twin Lakes sections and for an 870-foot reach fronting Cliff Drive in the Opal Cliffs-Capitola section are justified by prospective benefits, and that the public interest involved in protection of public property warrants Federal assistance in accordance with existing policy. The district engineer recommended adoption of a project by the United States authorizing Federal participation, subject to certain conditions, by contribution of funds in amount of one-third of the costs applicable to protection of the publicly owned shores of the problem area. Subsequent to completion of the district and division engineers' reports the policy of Federal aid in shore protection was modified somewhat by enactment of Public Law 826, 84th Congress, approved July 28, 1956. By separate communication the division engineer advised that consideration of the policy of Federal aid expressed in Public Law 826 indicated that the Federal share of cost for West Cliff Drive and for Cliff Drive in the vicinity of 49th Avenue should be increased.

In accordance with the policy expressed in Public Law 826, 84th Congress, the Beach Erosion Board recommended that projects be adopted by the United States authorizing Federal participation, subject to certain conditions, by the contribution of Federal funds in an amount equal to one-third of the first costs of work applicable to the publicly owned portions of the shores at West Cliff Drive, Twin Lakes Beach and Cliff Drive in the vicinity of 49th Avenue, plus 32.3 and 4.9 percent respectively of the first costs of work applicable to the privately owned shores at West Cliff Drive and Cliff Drive, all in Santa Cruz County, California. The Federal shares of the total project first costs under their present conditions of ownership are 32.7 percent for West Cliff Drive, one-third for Twin Lakes Beach and 23.5 percent for Cliff Drive. The projects for which Federal aid was recommended consist of the following.

a. At West Cliff Drive construction of a riprap seawall in nine segments with an aggregate length of about 2,885 feet;

b. At Twin Beach, construction of a protective beach with a berm width of 120 feet along the eastern and western segments by artificial placement of approximately 712,000 cubic yards of suitable sand, and construction of a stone groin at the east end of each of those two segments;

c. At Cliff Drive, in the vicinity of 49th Avenue, construction of a riprap seawall approximately 870 feet in length.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

STATE OF DELAWARE - KITTS HUMMOCK TO FENWICK ISLAND

The study area comprises the Delaware Bay shore from Kitts Hummock to Cape Henlopen, a frontage of about 32 miles with a general northwest-southeast orientation, and the Atlantic Ocean shore from Cape Henlopen to the State line at Fenwick Island, a frontage of about 24 miles with an approximate north-south orientation. The bay shore consists generally of a narrow strip of sandy beaches separating the bay from tidal lagoons and marshes. The ocean shore consists generally of sandy barrier beaches with well developed dunes. About 4 miles west and 13 miles south of Cape Henlopen, the shores are broken by Roosevelt and Indian River Inlets respectively which have been improved by dredging and jetty construction under Federal navigation projects. Mispillion River entering Delaware Bay has been similarly improved. The Delaware Bay shore is developed only at relatively small fishing and summer resort villages, such as Kitts Hummock, Slaughter Beach, Broadkill Beach and Lewes. Lewes Beach extending over 2½ miles from Fort Miles on Cape Henlopen to Roosevelt Inlet has a summer cottage development on the western half and fisheries and allied industries on the eastern half. The latter are on the harbor behind the breakwaters of the Federal Harbor of Refuge project. Fort Miles, a Federal military reservation, occupies about 1.5 miles of bay frontage and 2.7 miles of ocean frontage at Cape Henlopen. The ocean shore is developed at a number of summer resort communities, the principal ones being Rehoboth Beach and Bethany Beach. The permanent population of the coastal region of the study area is approximately 14,000. A large summer increase in population is drawn principally from the remaining area of the State, adjacent counties in Maryland and the cities of Baltimore, Maryland and Washington, D. C. In addition to the Fort Miles military reservation, Federally owned shore frontage comprises an undeveloped section 480 feet long at Broadkill Inlet and the two frontages at Lewes and Indian River Inlet occupied by Coast Guard Stations. Of the remaining shore frontage, about 38 percent is publicly owned and 62 percent privately owned.

The tides in the study area are semi-diurnal and have a mean range of 4.1 feet. The maximum tide is estimated to be about 9 feet above mean low water. The ocean shore is exposed to the unlimited fetch of the Atlantic Ocean. About three-quarters of the offshore waves approach from the direction northeast through east. The Delaware Bay shore generally between Broadkill and Slaughter Beaches is exposed to ocean waves entering the bay mouth. The bay shores farther north and south are exposed principally to waves generated in the bay, those to the north being protected by the southern New Jersey peninsula and those of the south being protected by the breakwaters of the Harbor of Refuge off Cape Henlopen. There is no apparent source of beach material to the shores of the study area other than erosion of the shores within the area. In Delaware Bay north of Mispillion River there appears to be little littoral drift. A nodal zone is located between Mispillion River and Roosevelt Inlet with predominant northward drift at Slaughter Beach, southward drift at Broadkill Beach and eastward drift at Lewes Beach. North of Bethany Beach the drift is predominantly northward toward Cape Henlopen, while south of Bethany Beach the drift is southward.

The district and division engineers and the Beach Erosion Board concluded that the most suitable plans of shore protection for the several sections of the problem area are:

a. For Kitts Hummock, widening 3,200 feet of beach to a berm width of 25 feet by artificial placement of sand;

b. For Slaughter Beach, widening 1,100 feet of beach to a berm width of 50 feet by artificial placement of sand and constructing two groins.

c. For Broadkill Beach, widening 1,000 feet of beach to a berm width of 50 feet by artificial placement of sand and landward extension of two existing groins;

d. For Lewes Beach, widening 4,600 feet of beach to a berm width of 50 feet by artificial placement of sand;

e. For the reach from Rehoboth Beach to Indian River Inlet, widening 2,500 feet of beach north of Indian River Inlet to a berm width of 50 feet by artificial placement of sand, including periodic placements of sand to provide continuing artificial nourishment to the shore northward to Rehoboth Beach;

f. For Bethany Beach, widening 4,800 feet of beach to a berm width of 50 feet by artificial placement of sand, and repairing seven and extending nine existing groins.

The district and division engineers and the Beach Erosion Board concluded that the plan for Rehoboth Beach-Indian River Inlet area is amply justified by evaluated benefits and that the public interest in the protective and recreational benefits warrant Federal participation to the maximum extent permitted by law. They recommended adoption of a project by the United States authorizing Federal participation, by contribution of Federal funds in an amount equal to the cost of protecting the Federally owned shore plus one-third of the costs for protecting the other shores from Rehoboth Beach to Indian River Inlet, Delaware (a total of 33.8 percent), by restoring 2,500 feet of beach immediately north of Indian River Inlet by artificial placement of approximately 130,000 cubic yards of suitable sand to provide a beach with berm 50 feet wide at an elevation of 12 feet above mean low water. Periodic placements of suitable sand to nourish and stabilize the shore from the inlet northward to the north limit of Rehoboth Beach in an estimated average annual volume of 145,000 cubic yards would be divided on the basis of need between a feeder beach immediately north of the inlet and one in the vicinity of Dewey or Rehoboth Beach, the initial placement of a feeder beach just north of the inlet to be made preferably in connection with the recommended beach restoration. The Federal share of the estimated annual costs of periodic nourishment are 33.8 per cent for a period not to exceed 10 years from the year of completion of the initial placement.

The Chief of Engineers concurred in the views and recommendations of the Beach Erosion Board.

COMPLETED COOPERATIVE BEACH EROSION STUDIES

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN		FEDERAL PROJECTS	
		H. DOC.	CONG.	RECOMMENDATION	AUTHORIZED BY CONGRESS
	ALABAMA				
Perdido Pass (Alabama Pt.)	18 Jun 54	274	84	Unfav.	
	CALIFORNIA				
Santa Barbara - Initial	15 Jan 38	552	75	Unfav.	
Suppl.	18 Feb 42				
Final	22 May 47	761	80	Unfav.	
Ballona Creek & San Gabriel R. (Partial)	11 May 38			Unfav.	
Orange County	10 Jan 40	637	76	Unfav.	
Coronado Beach	4 Apr 41	636	77	Unfav.	
Long Beach	3 Apr 42			Unfav.	
Mission Beach	4 Nov 42			Unfav.	
Pt. Mugu to San Pedro BW	27 Jun 51	277	83	Fav.	3 Sep 54
Carpinteria to Pt. Mugu	4 Oct 51	29	83	Fav.	3 Sep 54
Oceanside, Ocean Beach, Imperial Beach & Coronado, San Diego County	26 Jul 55	399	84	Fav.	
Santa Cruz County	13 Sep 56	179	85	Fav.	
Humboldt Bay (Buhne Pt.)	29 Mar 57			Fav.	
	CONNECTICUT				
Compo Beach, Westport	18 Apr 35	239	74	Unfav.	
Hawk's Nest Beach, Old Lyme	21 Jun 39			Unfav.	
Ash Crk. to Saugatuck R.	29 Apr 49	454	81	Fav.	17 May 50
Hammonasset R. to East R.	29 Apr 49	474	81	Fav.	3 Sep 54
New Haven Hbr. to Housatonic R.	29 Jun 51	203	83	Fav.	3 Sep 54
Conn. R. to Hammonasset R.	28 Dec 51	514	82	Unfav.	
Pawcatuck R. to Thames R.	31 Mar 52	31	83	Unfav.	
Niantic Bay to Conn. R.	11 Jul 52	84	83	Unfav.	
Housatonic R. to Ash Creek	12 Mar 53	248	83	Fav.	3 Sep 54
East R. to New Haven Hbr.	15 Nov 55	395	84	Fav.	
Saugatuck R. to Byram R.	14 Nov 56	174	85	Fav.	
Thames R. to Niantic Bay	17 June 57			Unfav.	
	DELAWARE				
Kitts Hummock to Fenwick Is.	11 Feb 57			Fav.	

LOCATION	BEB REPORT COMPLETED	H. DOC.	CONG.	FEDERAL PROJECTS	
				RECOMMENDATION	AUTHORIZED BY CONGRESS
	FLORIDA				
Blind Pass (Boca Ciega)	1 Feb 37	187	75	Unfav.	
Miami Beach	1 Feb 37	169	75	Unfav.	
Hollywood Beach	28 Apr 37	253	75	Unfav.	
Daytona Beach	15 Mar 38	571	75	Unfav.	
Bakers Haulover Inlet	21 May 45	527	79	Unfav.	
Anna Maria & Longboat Keys	12 Feb 47	760	80	Unfav.	
Jupiter Island	13 Feb 47	765	80	Unfav.	
Palm Beach (1)	13 Feb 47	772	80	Fav.	17 May 50
Pinellas County	22 Apr 53	380	83	Fav.	3 Sep 54
	GEORGIA				
St. Simon Island	18 Mar 40	820	76	Unfav.	
	HAWAII				
Waikiki Beach	5 Aug 52	227	83	Fav.	3 Sep 54
Waimea & Hanapepe Bay, Kauai	17 Jan 56	432	84	Fav.	
	ILLINOIS				
State of Illinois	8 Jun 50	28	83	Fav.	3 Sep 54
	LOUISIANA				
Grand Isle	28 Jul 36	92	75	Unfav.	
Grand Isle	28 Jun 54	132	84	Unfav.	
	MAINE				
Old Orchard Beach	20 Sep 35			Unfav.	
Saco	2 Mar 56	32	85	Unfav.	
	MASSACHUSETTS				
South Shore of Cape Cod (Pt. Gammon to Chatham)	26 Aug 41			Unfav.	
Salisbury Beach	26 Aug 41			Unfav.	
Winthrop Beach	12 Sep 47	764	80	Fav.	17 May 50
Lynn-Nahant Beach	20 Jan 50	134	82	Fav.	3 Sep 54
Revere Beach	12 Jan 50	146	82	Fav.	3 Sep 54
Nantasket Beach	12 Jan 50			Unfav.	
Quincy Shore	2 May 50	145	82	Fav.	3 Sep 54
Plum Island	18 Nov 52	243	83	Unfav.	
Chatham	22 Oct 56			Unfav.	

(1) A cooperative study of experimental steel sheet pile groins was also made, under which methods of improvement were recommended in an interim report dated 19 Sep 1940. Final report on experimental groins was published in 1948 as Technical Memo. No. 10 of the Beach Erosion Board.

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN		FEDERAL PROJECTS	
		H. DOC.	CONG.	RECOMMENDATION	AUTHORIZED BY CONGRESS
	MICHIGAN				
Berrien County (St. Joseph)	17 Jun 57			Fav.	
	MISSISSIPPI				
Hancock County	3 Apr 42			Unfav.	
Harrison County - Initial	15 Mar 44				
Harrison County - Suppl.	16 Feb 48	682	80	Fav.	30 Jun 48
	NEW HAMPSHIRE				
Hampton Beach	15 Jul 32			Unfav.	
Hampton Beach	14 Sep 53	325	83	Fav.	3 Sep 54
	NEW JERSEY				
Manasquan Inlet & Adj. Beaches	15 May 36	71	75	Unfav.	
Atlantic City	11 Jul 49	538	81	Fav.	3 Sep 54
Ocean City	15 Apr 52	184	83	Fav.	3 Sep 54
Sandy Hook to Barnegat Inlet	24 Mar 54	361	84	Fav.	
Review Report - Sandy Hook to Barnegat Inlet	6 May 57			Fav.	
	NEW YORK				
Jacob Riis Park, Long Island	16 Dec 35	397	74	Unfav.	
Orchard Beach, Pelham Bay, Bronx	30 Aug 37	450	75	Unfav.	
Niagara County	27 Jun 42	271	78	Unfav.	
South Shore of Long Island	6 Aug 46			Unfav.	
Selkirk Shores State Park	21 Oct 53	343	83	Fav.	3 Sep 54
Fair Haven Beach State Park	18 Jun 54	134	84	Fav.	
Hamlin Beach State Park	20 Sep 54	138	84	Fav.	
Braddock Bay State Park	15 Apr 55			Unfav.	
Fire Island Inlet to Jones Inlet	10 Feb 56	411	84	Fav.	
	NORTH CAROLINA				
Fort Fisher	10 Nov 31	204	72	Unfav.	
Wrightsville Beach	2 Jan 34	218	73	Unfav.	
Kitty Hawk, Nags Head & Oregon Inlet	1 Mar 35	155	74	Unfav.	
State of North Carolina	22 May 47	763	80	Unfav.	
	OHIO				
Erie County - Vic. of Huron	26 Aug 41	220	79	Unfav.	
Michigan Line to Marblehead	30 Oct 44	177	79	Unfav.	

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN		FEDERAL PROJECTS	
		H. DOC.	CONG.	RECOMMENDATION	AUTHORIZED BY CONGRESS
OHIO (cont'd)					
Cities of Cleveland & Lakewood	22 Mar 48	502	81	Fav.	3 Sep 54
Chagrin River to Fairport	22 Nov 49	596	81	Unfav.	
Vermillion to Sheffield Lake Village	24 Jul 50	229	83	Fav.	3 Sep 54
Fairport to Ashtabula	1 Aug 51	351	82	Unfav.	
Ashtabula to Penna.St. Line	1 Aug 51	350	82	Unfav.	
Sandusky to Vermilion	7 Jul 52	32	83	Unfav.	
Sandusky Bay	31 Oct 52	126	83	Unfav.	
Sheffield Lake V. to Rocky R.	31 Oct 52	127	83	Unfav.	
Euclid to Chagrin River	25 June53	324	83	Unfav.	
PENNSYLVANIA					
Presque Isle Peninsula, Erie (Interim)	3 Apr 42				
(Final)	23 Apr 52	231	83	Fav.	3 Sep 54
PUERTO RICO					
Punta Las Marias, San Juan	5 Aug 47	769	80	Unfav.	
RHODE ISLAND					
South Shore (Towns of Narragansett, South Kingstown, Charlestown & Westerly)	4 Dec 48	490	81	Fav.	3 Sep 54
SOUTH CAROLINA					
Folly Beach	31 Jan 35	156	74	Unfav.	
Pawleys Is., Edisto Beach & Hunting Island	24 Jul 51			Unfav.	
TEXAS					
Galveston (Gulf Shore)	10 May 34	400	73	Unfav.	
Galveston Bay, Harris County	31 Jul 34	74	74	Unfav.	
Galveston (Gulf Shore)	5 Feb 53	218	83	Unfav.	
Galveston (Bay Shore)	19 Jun 53	346	83	Unfav.	
VIRGINIA					
Willoughby Spit, Norfolk	20 Nov 37	482	75	Unfav.	
Colonial Beach, Potomac River	24 Jan 49	333	81	Fav.	17 May 50
Virginia Beach	25 Jun 52	186	83	Fav.	3 Sep 54

LOCATION	BEB REPORT COMPLETED	PUBLISHED IN H. DOC.	PUBLISHED IN CONG.	FEDERAL PROJECTS RECOMMENDATION	FEDERAL PROJECTS AUTHORIZED BY CONGRESS
	WISCONSIN				
Milwaukee County	21 May 45	526	79	Unfav.	
Racine County	5 Mar 52	88	83	Unfav.	
Kenosha	16 Sep 54	273	84	Unfav.	
Manitowoc County	15 Apr 55	348	84	Fav.	

CURRENTLY AUTHORIZED COOPERATIVE BEACH EROSION STUDIES

CALIFORNIA

STATE OF CALIFORNIA. Cooperating Agency: Department of Public Works, Division of Water Resources, State of California.

Problem: To conduct a study of the problems of beach erosion and shore protection along the entire coast of California. The current studies cover the Orange County and San Diego County Areas.

FLORIDA

PALM BEACH COUNTY. Cooperating Agency: Board of County Commissioners, Palm Beach County.

Problem: To develop the most economical means of restoring the beaches along the Atlantic Ocean shore of Palm Beach County to a satisfactory condition and protecting the restored beaches and shore property from future erosion.

KEY WEST. Cooperating Agency: City of Key West

Problem: To determine the best method of restoring and maintaining a protective and recreational beach fronting a 6,200-foot reach of South Roosevelt Boulevard.

FERNANDINA BEACH (AMELIA IS.) Cooperating Agency: City of Fernandina Beach.

Problem: To determine the best method for restoring and retaining an adequate beach for recreation and protection of oceanfront properties and improvements.

MASSACHUSETTS

COMMONWEALTH OF MASSACHUSETTS. Cooperating Agency: Department of Public Works.

Problem: To determine the most suitable methods of shore protection, prevention of further erosion and improvement of beaches. Current studies cover the shore from Pemberton Point to Cape Cod Canal in Massachusetts and Cape Cod Bays where development of specific plans are desired for protection of Crescent Beach, the Glades, North Scituate Beach, Brant Rock and Warren Cove; and from Cape Cod Canal to Provincetown in Cape Cod Bay where detailed study is desired for problems at Town Beach, Spring Hill Beach, East of Sesuit Harbor, Brewster Bluffs, Griffin Island, Pilgrim Beach, and shore immediately south of Hatches Harbor.

NEW JERSEY

STATE OF NEW JERSEY. Cooperating Agency: Department of Conservation and Economic Development.

Problem: To determine the best method of preventing further erosion and stabilizing and restoring the beaches, to recommend remedial measures, and to formulate a comprehensive plan for beach preservation or coastal protection. Current studies cover the Atlantic Ocean shore from Barnegat Inlet to Cape May Canal, Cape May Canal to Maurice River in Delaware Bay, and South Amboy to Shrewsbury River in Raritan and Sandy Hook Bays.

NEW YORK

SUFFOLK COUNTY (ATLANTIC COAST BETWEEN MONTAUK POINT AND FIRE ISLAND INLET). Cooperating Agency: Department of Public Works, State of New York.

Problem: To determine the most practicable and economic method of restoring adequate recreational and protective beaches and providing continued stability to the shores.

NORTH CAROLINA

CAROLINA BEACH. Cooperating Agency: Town of Carolina Beach.

Problem: To determine the best method of preventing erosion of the beach.

OHIO

MICHIGAN LINE TO MARBLEHEAD. Cooperating Agency: Division of Shore Erosion, Department of Natural Resources, State of Ohio.

Problem: To determine the best method of protecting the shores of the study area, including typical methods of protection for publicly and privately owned shores, especially to determine whether any changes should be made in recommendations contained in H.D. No. 177, 79th Congress in view of changed conditions and additional data; and to develop specific plans of restoration and protection of the shores of Metzgar Marsh, Crane Creek State Park and East Harbor State Park, and general plans for the protection of privately owned property.

PUERTO RICO

PT. SALINAS TO PT. VACIA TALEGA (SAN JUAN). Cooperating Agency: Department of Public Works, Commonwealth of Puerto Rico.

Problem: To determine most practical and economical method of preventing further erosion of the shore and stabilizing or restoring the beach, especially aimed to protect existing upland properties and future recreational, industrial or residential development areas.

RHODE ISLAND

SOUTH KINGSTOWN AND WESTERLY. Cooperating Agency: Division of Harbors and Rivers, Department of Public Works, State of Rhode Island

Problem: To develop the most suitable method of restoring and stabilizing the shores in connection with development of public beaches at East Matunuck in South Kingstown and Misquamicut Beach in Westerly.

TEXAS

ROLLOVER FISH PASS. Cooperating Agency: Game and Fish Commission, State of Texas.

Problem: To determine the best method of providing a stabilized pass at Rollover Inlet from the Gulf of Mexico into East Galveston Bay, including stabilization of adjacent shores.